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Operation of the U. S. Suction Dredge, "New Orleans"

The Federal government has had a large sea-going suction dredge of the Fruhling type in operation for about two years and interest in the results of its operations has grown steadily since it was completed. However, the dredge has not worked all this time under the conditions for which it was designed and few data have been published concerning it. It is of interest, nevertheless, to collect what information is available, but scattered through various reports of the district offices of the Engineer Corps, U. S. Army, under the direction of which the dredge is working.

When the Fruhling type of drag-head suction dredge was first introduced in this country, a description of several of the foreign vessels of this design was given in ENGINEERING NEWS, Mar. 5, and Apr. 23, 1908, with a review of the advantages of this dredge as a special tool for working in fine and free material.

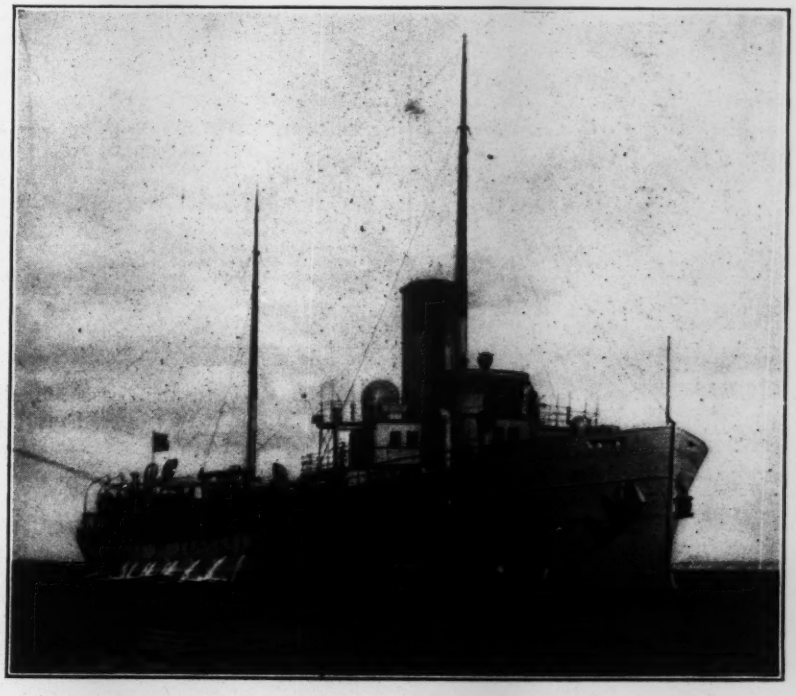


FIG. 1. U. S. DREDGE "NEW ORLEANS"

DESCRIPTION OF "NEW ORLEANS"—The dredge "New Orleans" was designed by the Fruhling Suction Dredge Co., of New York, to the requirements of the Chief of Engineers, U. S. Army, and was constructed by the Fore River Shipbuilding Co., of Quincy, Mass. Total cost was $538,087. The principal dimensions are given in the accompanying table:

TABLE I. U. S. DREDGE "NEW ORLEANS"; PRINCIPAL DIMENSIONS

Length overall, ft.	315
Length between perpendiculars, ft.	300
Breadth, molded, ft.	50
Depth, molded, ft.	26
Length of hopper, ft.	93½
Hopper capacity, cu.yd.	3000
Draft loaded, ft.	20
Width of suction head, ft.	12 to 18
Coal bunker capacity, tons	300
Speed loaded, knots	9

The vessel has a straight stem and twin semielliptical sterns, between which the dredging arm is located in the center plane of the ship. The hopper is located amidship and is divided into ten compartments, arranged in two parallel rows and separated by longitudinal bulkheads. The boiler room is located immediately forward, and the engine room abaft of the hopper. From the after engine-room bulkhead, extending aft, is the dredging well which divides the hull and forms the twin sterns. In this well is hung the dredging arm formed by a heavy girder 67 ft. long, to the outer end of which is secured the dredge head. The girder also carries two 9-in. pressure-water lines and two 26-in. suction pipes. The head is serrated along its scraping edge and high-pressure water is forced through sharp cutting teeth to aid in disintegrating the excavated material and to reduce it to suitable consistency for delivery by the pumps into the hopper. The girder is on trunnions at its forward end and is swung down by falls running from a high deck frame to the head. With this arrangement, the head can be raised clear of the water or lowered to a dredging depth of 50 ft. The suction connections with the interior of the head are made through hollow trunnions and the angle between head and girder can be adjusted by wedge pieces and chain screws.

The main engines comprise four sets arranged in tandem pairs on common bed-plates. They are of triple-expansion inverted marine type, having cylinders 12, 19 and 32 in. in diameter with a common stroke of 24 in. The total capacity is 2500 i.hp. Each of the two after sets of the main engines is coupled direct to one of the twin screw-propeller shafts and each of the two forward sets to a 26-in. centrifugal dredging pump. There are clutches, however, between the sets fore and aft so that the propelling and pumping engines may be coupled together to apply full power of two engines to each propeller when necessary. The two dredging pumps are of

special (Fruhling) design, the particular features being the small diameter of runner, the few large passages for material of extreme thickness, chambers lined with manganese steel, and removable wearing pieces of high-grade cast steel in the impellers. There are in the engine room also three vertical duplex pumps for supplying pressure water to the head jets and hopper pipes. These have a capacity of 300 cu.ft. per min. each. Steam for all this equipment is supplied by four water-tube boilers (Babcock & Wilcox semimarine), having a heating surface of 12,664 sq.ft. and a grate area of 317 sq.ft.

There are 20 hopper doors opened outwardly through the ship's bottom by means of four hydraulic rams. The doors may be operated individually or simultaneously for discharging the dredged material. The hopper has a water-pipe system with (1) sea connections which can be opened to let in mixing water for loosening the material which may have settled in the hopper and (2) pressure-

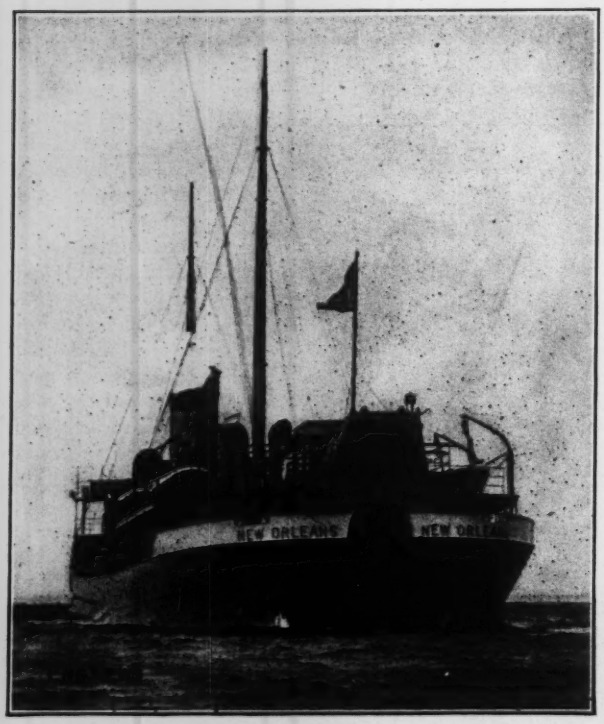


FIG. 2. STERN VIEW OF U. S. DREDGE "NEW ORLEANS".

water connections for clearing. Special attention has been given to the arrangement of settling troughs, baffle-plates and overflow sluices, so that surplus water may escape overboard with the minimum amounts of dredged material. A swivel discharge pipe is installed on deck and to this shore piping may be coupled; the hopper load may be pumped directly on shore or the material may be ejected from the swivel pipe into scows alongside.

ACCEPTANCE TRIALS—The contract with the Fruhling Co. provided for expert supervision during construction and a premium contingent upon the dredge maintaining for one month a pumping capacity of 4000 cu.yd. of mud per hour in Southwest Pass, Mississippi River. It was provided, however, that after the builder's steam trials, the dredge was to have a preliminary test in sand and mud on the Delaware, Chesapeake or other suitable location mutually agreed upon. Here, the vessel was to dredge 2000 cu.yd. per hour in sand and 4000 in mud.

The conditions for measuring capacity were agreed on as follows: the sand was to be clear, coarse material measured as settled in the hopper; mud was to be such as was regularly deposited in river channels, free from hardening or caking, to be measured by sampling the dredgings at about half depth of full hopper and letting the samples stand to determine the percentage of water contained, which percentage was to be deducted from the measured quantity of mud in hopper.

The sand-dredging (deep-sea) trials were carried out in Cape Cod Bay under adverse weather conditions and with an untried crew. The dredge pumped 1794 cu.yd. of coarse sand in 35 minutes, equivalent to a rate of 3075 cu.yd. per hr. Next, the dredge was tested for mud in the River Delaware, pumping 2650 cu.yd. in 26 minutes, equivalent to a rate of 6115 cu.yd. per hr.

On arrival at Southwest Pass, in March, 1912, it was found that the extraordinary floods of that year had scoured out the enormous quantities of muck usually found in the channel above the jetties and it was necessary to wait for six months, until November, 1912, before the mud trials could be carried out.

The reports of this trial operation show 118 loads distributed over 21 actual working days. The data yielded are given in Table II:

TABLE II. RESULTS OF ACCEPTANCE TRIALS; U. S. DREDGE "NEW ORLEANS"

Total quantity for month, cu.yd.	301,530.8
Av. quantity per load, cu.yd.	2,565.4
Max. quantity per load, cu.yd.	2,852.0
Min. quantity per load, cu.yd.	1,996.4
Time of loading (13 loads), min.	15
Time of loading (105 loads), min.	20
Average capacity mud per hr., cu.yd.	7,891.6
Max. capacity (15 min. pumping), cu.yd.	10,998.8
Max. capacity (20 min. pumping), cu.yd.	8,556.0
Min. capacity (20 min. pumping), cu.yd.	6,417.0
Min. capacity (25 min. pumping), cu.yd.	7,746.9
Av. depth of dredging, ft.	36
Max. depth of dredging, ft.	45
Min. depth of dredging, ft.	30

It was found, in the Mississippi River trials, that when the suction head encountered masses of compressed mud, such as occurs when the underlying soft strata are pressed up through the overburden of sand deposited by the river, that the dredged material was discharged into the hopper so thick that it had the viscous appearance of molasses or pitch and would barely run from the discharges.

It was found, as noted, that the average capacity of the dredge was 7891.6 cu.yd. of mud per hour, representing 82.5% as mud. As a matter of interest, it was desired to know how much of this was solid dry material. On sampling and removing the water, the dry material constituted 37.5% by weight and 42.7% by volume. Based on the percentage by volume the actual amount of dry material per average load was 1091.2 cu.yd., and the average capacity per hour for dry material was 3369.7 cu.yd.

The general character of material where dredging was badly needed was hard sand and to dredge mud for the test, localities had to be chosen where the depths were greater and dredging not so urgently needed at the moment.

SERVICE OPERATIONS—While awaiting the acceptance trials, the "New Orleans" operated 105 days from June to October, 1912, about Southwest Pass, in fine hard sand or in sand and silt mixed. In these shoals, the pumping capacity developed was 609 cu.yd. per hr., owing to ex-

tremely compact top surfaces permitting very little bite of the dredge head.

It should be noted here that the action of the scraper head in free sand is quite different from the action in mud. It is not found desirable in sand that the head should penetrate beyond a few inches below the surface, otherwise the propulsion of the vessel is retarded and maneuvering abilities reduced. The action in sand is therefore more of a scraping or gathering up by the head of loose sand so that the inrush of water to the suctions may encounter a pile of free material to increase the percentage delivered over that with the ordinary suction type of dredge. To assist this action, baffle-plates are inserted in the forward opening of the dredge head so as to compel the inrush of water to take place close to the cutting edge and through the sand scraped up in front of the head.

For conditions such as encountered in 1912 in Southwest Pass, it is considered that some means must be devised to let the head burrow beneath the sand surface so as to reach the softer underlying strata. A screw cutter is being developed by the Fruhling Co., which can be detached when not needed. This would be driven by a cutter engine on deck working through shaft and gearing on the dredging arm. Meantime, study is being conducted by the Engineer Corps (1) with a short head of less maximum bite and (2) by changing the head to one of grating type with small teeth added for loosening up the hard stratum.

For handling soft clay, the head is reduced in width by removal of the end segments and cross-bars with jets are fitted across the trunnion orifices. Without such provisions, there is a tendency of clayey material to throttle in the suction orifices, the broad head cutting off more than can be removed. Arrangements are being worked out for a series of rotary saw blades threaded on a spindle arm carried across the head parallel to the cutting edge and suction opening and rotated by shaft and gearing from the cutter engine on deck. Preliminary tests have shown that this should extend the scope of this type of dredge and enable it to handle some forms of refractory clay for which previously it has not been advocated.

After trials and up to the end of the fiscal year, June 30, 1912, the dredge operated during daylight hours only for 47 days and removed 186,550 cu.yd. of material. The cost per cu.yd., etc., is not given for this period since the work was largely experimental, trials being made of the efficiency under different arrangements of the changeable parts of the dredge head.

Even under adverse conditions the "New Orleans" has developed an average pumping capacity of 768.8 cu.yd., due to the hard bottom encountered, instead of that for which the dredge was designed. The cost of removal is reported at 6.65c. per cu.yd. exclusive of extraordinary repairs and 7.25c. including that item, which is not far from the performance of other dredges. This is shown by Table III.

TABLE III. PERFORMANCE OF U. S. DREDGES WORKING IN SAND, MUD OR SILT, 1912-1913

Dredge	Locality	Total cu.yd.	Material	Cost, cents per cu.yd.
Atlantic	New York	960,296	Sand, etc.	11.1
Benyaurd	S. W. Pass	1,231,935	Sand, mud, silt	7.1
Burton	Great Lakes	299,712	Sand, mud, clay	10.3
Cape Fear	North Carolina	646,166	Sand, clay, mud	8.3
Caucus	Florida	718,938	Sand, shells	8.5
Charleston	Mobile	457,172	Sand, mud	7.2
Chinook	Columbia River	80,423	Sand	11.7
Clatsop	Columbia R.	2,244,602	Sand	3.4
Comstock	Texas	78,753	Sand, clay, silt	37.5
Delaware	Philadelphia	5,514,753	Mud, sand	1.8
Galveston	S. W. Pass	1,615,265	Silt, sand	6.6
Gedney	Mass.	134,828	Mud, sand, gravel	28.1
Hedney	R. I.	55,405	Sand, shells	7.4
Key West	Florida	585,246	Sand, shells	10.0
Manhattan	Philadelphia	3,985,487	Mud, sand, gravel	2.4
Navesink	N. Y.	501,678	Sand	9.4
Raritan	N. Y.	1,849,355	Sand	5.9
	Hudson River	199,988	Mud, debris	10.2
Sabine	Tex.-La.	375,565	Mud, silt	9.4
St. John	Florida	133,863	Sand, shells	4.7
Sumter	S. Carolina	940,997	Sand, mud	7.6
Winyah Bay	Charleston	438,428	Sand, mud	9.9

The Pennsylvania State Bureau of Township Highways

In an endeavor to build and maintain good country roads without excessive expenditure, the State of Pennsylvania has established under its State Highway Department a Bureau of Township Highways, which is in charge of a Deputy Highway Commissioner designated by the State Highway Commissioner.

This Bureau of Township Highways has general supervision of all township highways and bridges which are constructed, improved or maintained, in whole or in part, by state funds, excepting state and state-aid roads otherwise provided for.

In conformity to this scheme, the state has been divided into districts each in charge of a Superintendent of Highways. This official is to visit and inspect, at least once a year, all the roads and bridges of his district and is to advise and direct how to make repairs, maintenance and improvement work. All plans and specifications and estimates for township bridges, culverts and road work must be approved by the District Superintendent of Highways.

The general supervision of road work in townships of the second class is in the hands of three Township Supervisors, who are elected to serve six years. The supervisors of each township meet annually and levy a road tax, which is not to exceed 10 mills for each dollar of valuation, without an order from the court of quarter sessions, in which case the tax may be larger. The township supervisors are instructed in their duties by the State Highway Commissioner through bulletins.

As soon as the township supervisors of each township are organized, they must divide their township into one or more road districts and employ a superintendent or roadmaster for each district, who is paid by, and is responsible to the township supervisors, but is subject to the rules and regulations of the State Highway Commissioner. The roadmasters have authority to employ and hire laborers, teams and equipment, making a monthly report to the township supervisors. The supervisors only have the authority to purchase equipment and materials, but all such contracts in excess of $100 must be approved by the District Superintendent of Highways.

One of the interesting features of the township scheme is the provision for county associations of township supervisors and commissioners, the expenses of which are paid for out of the public funds. These county associations are to hold annual or semiannual conventions at the various county seats for the purpose of considering and discussing road work. It is a part of the official duty of the township supervisors and the commissioners (of towns of the first class) to attend these conventions, their expenses being paid by the county. It is a part of the official duty of the State Highway Commissioner or his representatives to assist in the formation of these associations and to be present at their deliberations.

Concrete Protection Pier for a Railway Under-Crossing

We discussed in our editorial columns last week, the value of masonry guard piers adjacent to a bridge portal to protect the trusses from being struck by derailed cars. The accompanying drawing shows a bridge erected a couple of years ago in Cleveland, Ohio, at a point where the tracks of the Cleveland Short Line Ry. are carried across the main line and three side tracks of the Nickel Plate R.R.

As shown in the drawing, the columns which carry the overhead structure are protected from the impact of a derailed car by a substantial protection pier, 3 ft. in thickness and 20 ft. long, with the top about 10 ft. above the rail level. This concrete pier is reinforced with old rails, embedded in the base, 5 ft. long and 2 ft. apart.

Besides the protection afforded by these guard piers, the columns themselves are embedded in concrete and a heavy concrete wall extends the entire length between the columns. Besides its interest in connection with these collision piers, the structure is worth study by reason of the thorough protection given to the overhead structure from locomotive gases. The floor is of reinforced-concrete construction, the stringers being entirely embedded in concrete and the panels of concrete between the stringers being heavily reinforced. The concrete is also carried well up on the girders and is extended out on the approaches beyond the girder itself so as to furnish here also some protection from the shock of a derailed car. For the privilege of reproducing this drawing, we are indebted to the courtesy of B. R. Leffler, bridge engineer of the Lake Shore & Michigan Southern Ry.

The drawing certainly deserves the careful study of engineers who are responsible for the design of railway

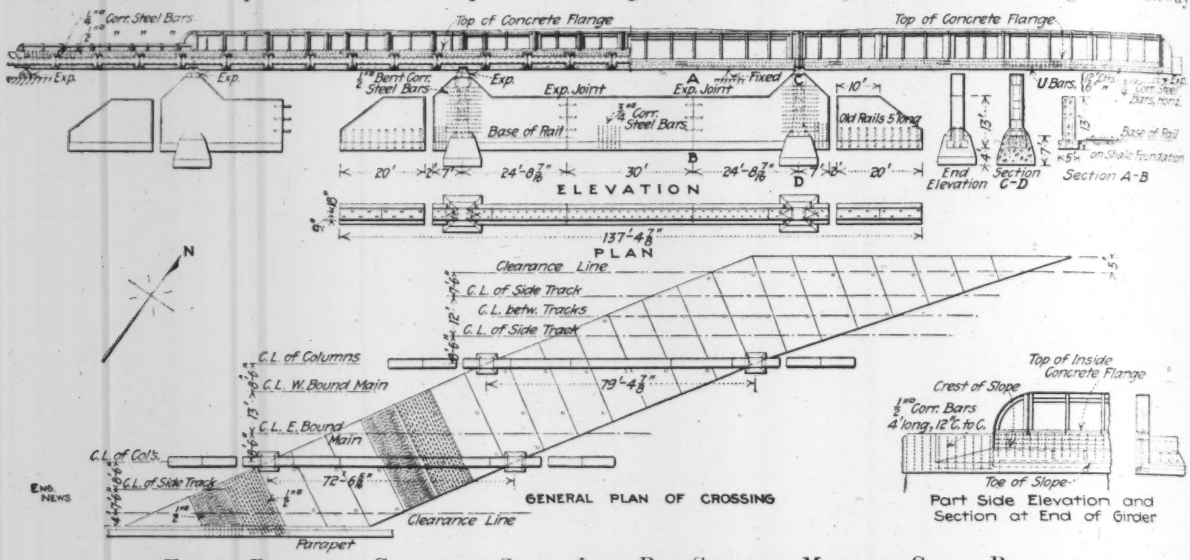
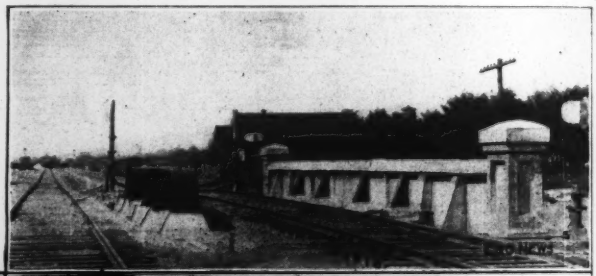


FIG. 1. BRIDGE ON CLEVELAND SHORT LINE RY., SHOWING MASONRY GUARD PIERS



FIGS. 1-4. VIEWS OF HEAVY CONCRETE PEDESTALS SUBJECTED TO TRAIN WRECK

under-crossings and in fact of all railway engineers. A derailment which knocks out the columns supporting an overhead structure might not only let this structure fall and crush a passenger car crowded with people, but might also cause a train approaching on the upper tracks to dash into the chasm left by the fall of the bridge, should the fall occur just as a train on the upper track approached the bridge.

That such substantial masonry barriers at the ends of bridges may serve a valuable purpose in saving bad wrecks, will surely be generally agreed by engineers. As illustrating how even a massive concrete pillar at a bridge portal may serve to stop a derailed car and prevent its plunging over the side or into the opening left by the fall of the bridge, a correspondent sends us three photographs, herewith reproduced, showing a wreck which took place Dec. 5, 1911, at the Mayfield Road bridge in Cleveland, Ohio.

A derailed car swerved to one side as it approached the bridge and struck heavily against the concrete pillar at the bridge portal. The pillar, with the masonry back of it, withstood the shock and brought the car to a stop. Had the structure been an ordinary deck bridge without parapets, there can be little doubt that the car would have gone over the side and fallen into the street with possible heavy resulting casualties.

It is probable, from the comparatively small amount of damage which the car suffered, that it was moving at low speed when the collision occurred, otherwise the damage to the structure also would have been much more serious. If, however, a concrete protection pier had been built in advance of the structure, like those shown on the drawing of the Lake Shore bridge, the pier would have absorbed the impact even if the train were running at good speed and protected the bridge from injury.

A Civil Engineers' Strike that Never Came off

By Willard Beahan*

We were building the Texas & Pacific Railroad in the early '80's between Fort Worth and El Paso, a distance of 700 miles. In January, '81, the grading was as far west as 240 miles out of Fort Worth and was reaching there the Llano Estacado, or Staked Plains. The grading that winter was light. This fact and a cold winter probably caused the Assistant Engineers—our title for men in charge of ten-mile sections of grading—to become restless, so they concluded to strike.

The men at the west end drew up a declaration to the effect that if the pay of Assistant Engineers was not raised to a certain figure by April 1, each and all would go out on a strike. The plan was to pass this war document down the line to each Assistant Engineer successively, with a request for him to sign it and forward it to his next neighbor to the east. At the time the paper reached me, I had just received my promotion to Assistant Engineer. I did not sign it, but I wrote a letter explaining that I was satisfied with my pay, attached the letter to the declaration of war and sent it on as requested. It so happened that in February I was sent around to El Paso to try the civilizing effect of a railroad on the Apache Indians.

*Lake Shore Bldg., Cleveland, Ohio.

A year passed, and I returned East. On meeting Capt. W. W. Dentler, one of the assistant engineers, who had signed the paper, he told me the outcome. The paper was addressed to Maj. D. W. Washburn, the Chief Engineer, going over the head of our Division Engineer, very improperly. When the first of April came the men had been shifted, advanced in pay and conditions changed. They got nervous about April 1, but they heard nothing from the Chief Engineer. They regretted what they had done. They wanted to get hold of that paper. So another paper was started after the first. The trail stopped short at the tent door of Michael O'Grady, a grand old Irishman, a graduate at the University of Dublin, and who came to this country as a civil engineer in 1854. Capt. Dentler, my informant, was sent by the others to interview Mr. O'Grady. The following conversation ensued:

"Mr. O'Grady, did you receive last January a declaration to the Chief Engineer, we started from the west end for each Assistant Engineer to sign and forward?"

"I did, Captain Dentler."

"Did you sign it, Mr. O'Grady?"

"I did not, sir."

"Did you send it on?"

"No sir, I filed it."

"Ah!" (with satisfaction). "Then since you received it and did not forward it, but filed it, won't you return it to me so I can send it to the men who signed it, so that each can tear his name off as they now wish to do?"

"Now my dear sir, Captain Dentler, if you please, I was surprised and much grieved when I got that paper. The young gentlemen must have signed it thoughtlessly. It was most unprofessional in the extreme. We are not laborers. We represent the company. It is mutiny. It is desertion in the presence of the enemy. If any Assistant Engineer is dissatisfied, let him speak to the Division Engineer. If he gets no satisfaction, let him then write to the Chief Engineer. Major Washburn is a most honorable gentleman, and I have known him for years. Now, sir, in the interests of the good name of the profession, I filed that paper and said nothing. Gladly would I return it to you in your present frame of mind, but I am unable to do so. For I filed it, sir, in my stove and there was fire in it at the time."

The Railroad Water-Supplies of Minnesota have been investigated by the Minnesota State Board of Health and 32 of 61 sources have been condemned as either polluted or liable to pollution. The investigation followed the promulgation of a regulation of the Secretary of the Treasury (Public Health Service), issued Jan. 25, 1913, to the effect that water provided for passengers by interstate common carriers must be certified by state or municipal health authorities "as incapable of conveying disease." The Minnesota State Board of Health decided to make a systematic field inspection and analysis of the water-supplies used in both interstate and intrastate traffic. A report on the investigation appears in "Public Health Reports" (Washington, D. C., U. S. Public Health Service), for May 15, 1914. An abstract follows:

(1) The results of this investigation show that 52.45% of the supplies investigated could not be recommended in their present state as drinking water-supplies.

(2) Of the 32 supplies on which recommendation was refused, 31 were pronounced unsafe by the field investigation and of these but 16 showed analytical evidence of pollution. In one instance only did the analytical results alone constitute the refusal for recommendation.

(3) A careful and adequate field investigation in addition to the analytical determinations is required before a satisfactory opinion should be ventured concerning the safety of a given water-supply.

(4) The owners of practically all the condemned water-supplies can convert these into safe supplies at a relatively small cost by following in detail the recommendations given each locality.

Experience with Bituminous Pavements, Charlotte, N. C.

By S. H. Lea*

The city of Charlotte, N. C., has a population of about 50,000 and covers an area of 12.8 sq.mi. Within the city limits, there are about 125 mi. of streets laid out, of which about 96 mi. have been opened up for public use. Of the total mileage of streets in use, 25.4 mi. are paved, 30 mi. are macadamized, and 40 mi. are graded and partly graded but not otherwise improved. The extent of paving work that has been done in the city is shown in the accompanying tabulated statement.

Kind of paving	When laid	Area sq.yd.	Cost per sq.yd.	Grading cu.yd.	Remarks
Bitulithic	1907-09	40,422	$2.15	$0.49	Price includes 25c. sq.yd. royalty
Bitulithic	1912-13	85,526	1.69	0.45	
Bituminous macadam	1912-13	8,843	1.31	0.39	Penetration method
Asphaltic concrete	1912-13	54,694	1.34	0.38	
Sheet asphalt	1907-12	35,777	1.70	0.54	
Sheet asphalt	1912-13	67,529	1.59	0.38	
Sheet asphalt	1913-14	53,871	1.54	0.38	
Brick	1907-12	14,276	2.56	0.45	St. Ry. tracks
Granolithic	1907-13	59,674	1.71	0.39	St. Ry. tracks

Brick and granolithic pavements were laid by the street-railway company along its tracks wherever these traversed paved streets. The area so paved comprises a strip covering the space between the rails and extending 14 in. outside either rail. Such an arrangement has not proved entirely satisfactory, since it involves the use of two different types of pavement in juxtaposition on a part of the street where traffic is heaviest. Experience here has shown that there has been considerable wear along the longitudinal joints, resulting in much unevenness of the surface adjacent to the joint.

Construction Methods

Sheet Asphalt—Under existing contracts, the city is paving about 80,000 sq.yd. of roadway with sheet asphalt on a concrete base. The contractor has an equipment consisting of a one-car steam-operated asphalt plant, 14 Troy dump-wagons, one 10-ton steam-roller and one 5-ton tandem roller. This outfit handles the work satisfactorily and good progress is made, the work being done in a thorough and satisfactory manner.

The concrete base, 4 in. thick, is composed of a 1:3:6 mixture, mixed rather wet, and with the surface slightly roughened. The concrete is allowed to season several days, is then swept clean and the binder course laid. This is composed of stone and asphaltic cement, the stone not exceeding ¾ in. in its largest diameter, and of varying size down to dust. The ingredients are heated and thoroughly mixed by machinery, from 5% to 8% of asphalt being used. The resulting binder is then taken from the mixing plant to the street in canvas-covered wagons at a temperature between 200° F. and 325° F. The mixture is distributed over the concrete base and rolled to a depth of not less than 1 in.

The surface mixture is brought to the street at a temperature between 230° F. and 350° F. It is dumped outside the space to be covered, deposited roughly in place by means of hot shovels, and uniformly spread by means of hot iron rakes. It is then compressed by a tandem roller until the surface conforms to the established grade and the top coat has a thickness of not less than 2 in. A small amount of hydraulic cement is then swept over the surface, and a space of 18 in. next to the curb is painted with hot, liquid asphalt.

Inspection

An inspector is stationed at the asphalt plant and one is kept on the street where pavement is being laid. Samples of paving material are taken during each day's run and analyzed by a chemist employed by the city. Daily reports are made by each inspector and by the chemist; these reports are kept on file by the city engineer.

The plant inspector's report comprises a complete statement of material prepared at the plant during the day's run, covering the following items: Number of boxes of binder and of surface material prepared, quantity of flux and of asphalt used, penetration of asphaltic cement, box weights of stone, sand filler and asphaltic cement. Also a statement of sand siftings showing proportion of different sizes of sand, and the temperatures of sand and surface mixtures, taken at frequent intervals. The report shows name of street and locality where the material is used, also total quantities of material sent from plant to street for use.

The paving inspector on the street where pavement is being laid, makes a daily report which contains the following information: Area of concrete base laid, area of

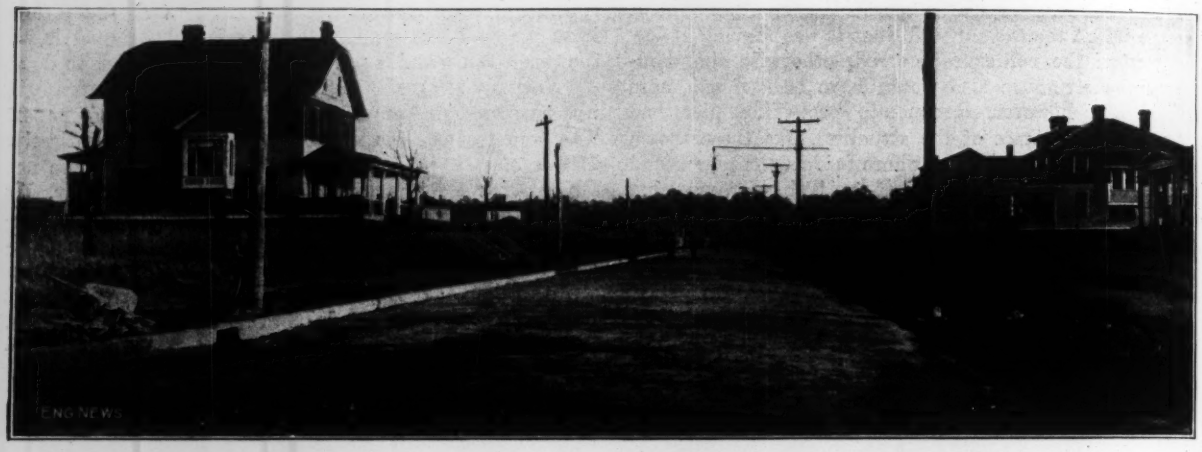


Fig. 1. Concrete Base, 4 In. Thick, for Asphalt Pavement on Residence Street, Charlotte, N. C.

*City Engineer, Charlotte, N. C.

binder laid or area of wearing coat laid; also minimum thickness of each course laid during the day. He is provided with a suitable thermometer with which to take the temperature of each load of paving material as it is delivered on the work; and he is required to reject all material that is overheated or too cold for use.

The chemist analyzes the samples taken during each day's run and submits a daily report of his work to the city engineer. The report shows percentage of bitumen soluble in carbon bisulphide, percentage of the various sizes of sand or filter, the specific gravity of sample tested, and the penetration or degree of hardness of the asphaltic cement at 77° F.

The chemical analyses and tests are made in a laboratory established in the city. In this way, all tests are made promptly and results are known before the paving work advances too far to correct quickly any defects that may be discovered in the material used.

Maintenance

All paving in the city has been laid under a five-year maintenance guarantee. The earlier work was done in 1907-08 under an agreement requiring that the contractor shall:

So construct the pavement that it shall be and remain in good repair and condition for and during a period of five years from the final completion of the work, so that at the end of said period said pavement shall be in good, serviceable condition and free from any defects that will impair its usefulness as a roadway.

This particular job went out of guarantee period last year and, just before the end of the five-year period, the contractor did considerable patching and resurfacing, leaving the pavement in what was claimed to be good, serviceable condition. The city, represented by its Executive Board, took a different view of the matter and claimed that many decided irregularities and depressions in the surface, especially along the car tracks adjacent to the brick paving, constituted a poor, unserviceable condition and refused final acceptance of the work or release of the maintenance bond. Litigation has resulted and the matter is now in the courts.

The later paving work has been done under different specifications. The present requirements are that:

The contractor shall keep and maintain the pavement in repair for the space of five years from its final acceptance. At any time after the acceptance of the work, or before the expiration of maintenance period that the city may require, all cracks or broken places to be cut out, refilled and repaired with the proper material, and all depressions raised to the proper level and all bunches reduced to the proper level. All repairs other than those due to opening of street by other parties, shall be made by the contractor at his own expense. All decayed or decomposed pavement must be taken out, and all cracks cut and filled with proper material. All depressions that measure more than ½ in. from the under side of a straight-edge, 4 ft. long, held on any part of the surface, shall be refilled to the proper grade.

Cost Data

As shown by the tabular statement, the cost of paving work in this city per sq.yd. has ranged from $1.31 for bituminous macadam to $2.15 for bitulithic, the latter price including a royalty charge of 25c. per sq.yd. The cost of grading per cu.yd. ranged from 38c. to 49c. Common labor costs $1.25 per day, and a team with driver costs from $3.50 to $4.50 per day. Crushed stone costs from $1.65 to $1.95 per cu.yd., delivered on the work, and sand is delivered for $1 per cu.yd.

During January of the present year, some repair work was done upon the pavement that went out of guarantee last year and which is now in litigation. The repairs were scattered over a wide area and the work was done by the contractor for the city at actual cost plus 15%. Sheet asphalt was used for patching, that being most convenient because it was readily available.

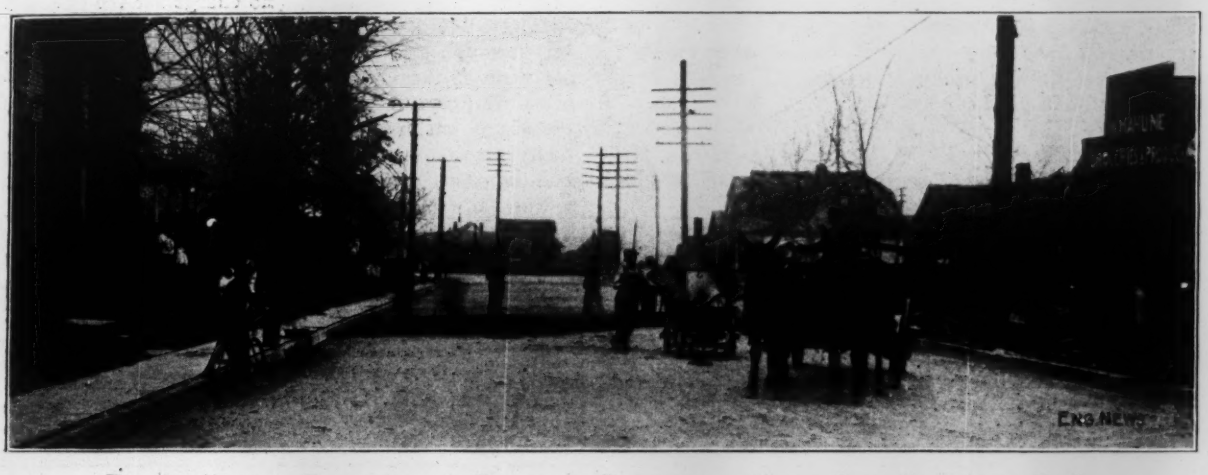


Fig. 2. Asphalt Pavement for Residence Streets of a Small City, Charlotte, N. C.

The total area covered by patches was 308.6 sq.yd. The price paid for material and labor was $609.75; this made the cost per sq.yd. of area actually covered, $1.976.

Conclusions

While there has not been sufficient time during the few years of our paving operations for conclusive results to be obtained, still the experience thus afforded has furnished some valuable information concerning the relative merits of different paving materials used under local conditions.

The first pavements were laid upon a macadam base and this form of construction has proved unsatisfactory. It has been found that where worn or broken spots occur in the surface, disintegration proceeds more rapidly over a macadam base than over one of concrete. The yielding, springy nature of the macadam allows considerable verti-

cal motion under the impact of traffic and this results in rapid raveling action.

The bituminous-macadam pavement constructed by the penetration method has not been considered a success. For a considerable while after its completion this pavement was in poor condition, with very little surface bond. Later, however, conditions improved and, under the action of warmer weather and traffic, the pavement became firmer and more compact, although the surface is still rough. All pavement in this city is now laid upon a concrete base, and sheet asphalt is being used on most of the streets.

PERSONNEL—The paving contractor is R. M. Hudson, of Atlanta, Ga., for whom T. B. Gillespie is Superintendent; the chemical analyses and tests are made by Frank P. Drane, of Charlotte. F. J. Blythe is Assistant Engineer in immediate charge of construction, the writer being City Engineer.

Creosoted Piling in Galveston Bay Bridge

BY F. B. RIDGWAY*

SYNOPSIS—The most valuable guide in treating timber with preservative is long-time records of the behavior of treated timber. Creosoted piles in the Santa Fé Railroad bridge across Galveston Bay have been in place for 38 years for the original bridge and 18 years for the newer bridge. The results of an expert examination of these piles show that the piles 38 years old, which received a very light treatment of creosote are mostly destroyed. Of the 18-year old piles, about 3000 in number, which were given a heavy treatment of creosote, over 90% are still in good condition.

In 1875, the Santa Fé Ry. let a contract to a Mr. Trundey for a number of creosoted Southern yellow-pine piles to be used in the construction of a trestle across Galveston Bay. The late J. W. Byrnes, who afterward became General Manager of the International Creosoting & Construction Co., and who had previously had some experience in the manufacture of creosote oil and in the treatment of timbers, was employed by Mr. Trundey to take charge of the treating operations.

No records of treatment as to the method of seasoning the piles have been preserved. It is known that the contract called for a light treatment, but the actual amount of oil injected, if it were known at the time, has not been recorded, though it was probably something between 8 and 12 lb. of oil per cu.ft. of timber. Mr. Byrnes protested against this light treatment, not believing that the result would be satisfactory and fearful of the bad effect it might have on the creosoting industry. During the absence of Mr. Trundey in New York, Mr. Byrnes treated a number of these piles with a heavy charge of oil, the maximum amount which could be injected with the facilities then in use. Upon his return, Mr. Trundey was very much provoked at what he thought to be a ruthless waste of oil and Mr. Byrnes, who was at the time also in the creosote oil business and was furnishing the oil for this work, agreed to make no charge for the excess of oil injected into these piles.

A thorough examination of these piles would indicate that none of them were as thoroughly penetrated or received as heavy a charge of oil as those subsequently treated and put in the bridge in 1895.

The results from these 1875 piles were sufficiently good, however, to warrant the use of creosoted piling in the reconstruction of the trestle which was done in 1895. Though it is a matter of record that in 1895 a great many of these early piles were still good, none of them were used in the new work but they were not taken out and all were left standing between the bents of the new structure.

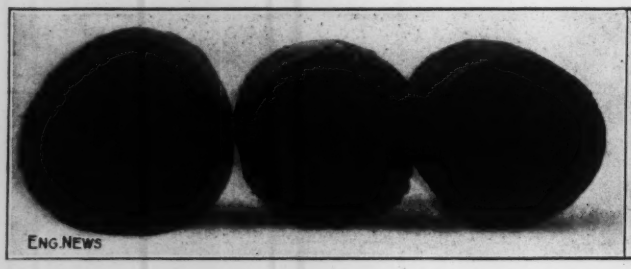


FIG. 1. AIR, WATER AND MUD SECTIONS CREOSOTED PILE NO. 1, TREATED 1875

FIG. 2. AIR, WATER AND MUD SECTIONS CREOSOTED PILE NO. 2, TREATED 1875

In July, 1912, after the completion and opening of the Galveston Causeway, the Santa Fé began to dismantle the railroad trestle, but the work was not completed until about the middle of 1913. At this time, the remains of the old 1875 piling were also taken out.

These 1875 piles, by virtue of the fact that they were left standing in the water until the present, have served as a test for 38 years. On examination after removal, it was found that the sections of these piling above the water line were very light in color and had almost entirely lost every outward appearance of ever having been creosoted. The heart of the stick of the air sections was, in many cases, very badly decayed though the outer por-

*In charge of Department of Chemistry and Tests, International Creosoting & Construction Co., Galveston, Tex.

tions were sound. In other cases, even though no more than 50% to 60% of the stick had been penetrated by the oil, the heart, as well as the outside portions of the stick, was sound. This difference in condition is due to the fact that in certain of these piles, sufficient of the butt of the pile was cut off after driving to expose the untreated center of the stick and in consequence, decay started. In other cases, the piles had been driven sufficiently deep so that it was unnecessary to cut further from the end of the pile than the oil had penetrated and in consequence no untreated wood was exposed and there was no decay.

The water sections of the 1875 piles were, in the majority of cases, more or less attacked by the teredo and limnoria. In many cases, this section of the pile was almost eaten away. In a few cases, the water section was in a practically sound condition. The action on the piles is due to both teredo and limnoria; but where the piling is badly eaten away, it is difficult to determine just what effect is due to each as the limnoria largely destroys the trail of the teredo. It is only in recent years that any marked attack by limnoria has been noted in the waters of Galveston Bay. The greatest attack is just below the water line but in many cases on these old piling, the attack extends clear to the mud line.

It seems to be the common belief, that in waters where the limnoria works, its action is more severe and more difficult to retard than that of the teredo. The writer does not possess sufficient data to either confirm or deny the above belief; but it must be borne in mind that after a stick of timber has been honeycombed by the teredo and the wood structure almost completely torn down, it takes but little effort on the part of the limnoria to finish the job and get the credit for the whole thing. The writer noted a number of instances where this appeared to be the case and a close examination of a great many piles revealed the fact that the teredo had progressed further into the stick than the limnoria.

Fig. 1 shows three sections of one of the good 1875 piles (Pile No. 1). The sections shown from left to right are respectively, air, water and mud sections of the pile. The water section is taken just below the water line at the point of greatest activity of the marine borers. The other two sections are taken at sufficient distances from the ends of the stick to show only radial penetration of oil and to be free from the longitudinal penetration from the end of the stick. This stick is approximately 14 in. in diameter, shows a 3-in. penetration of oil and an 8-in. untreated center, the line of demarcation between the treated and untreated wood being very sharp.

Fig. 2 shows three similar sections of one of the 1875 piles (Pile No. 2), which was almost entirely eaten away. This pile is approximately 11 in. in diameter, shows a 3½-in. penetration of oil and a 4-in. untreated center. The line of demarcation between the treated and untreated wood is in this case also very sharp.

The result of analyses of both piles as well as method of conducting same are given in the table.

Before entering into any discussion regarding the analyses or their significance, the writer wishes to call attention to a number of reasons why these piles should show a larger amount of oil present in the mud section of the pile than in the sections above it and why the results of the analyses do not show directly the treatment given the piling.

1. A certain amount of evaporation and leaching out

TABLE I. ANALYSES OF CREOSOTE OILS FROM GALVESTON BAY BRIDGE PILES

Analysis No.	Description of sample used for analysis	Amt. oil per cu.ft. extracted lb.	Est. treatment whole pile lb.	Treatment given chg. material lb.	Distillation of extracted oil								Condition of distillates at 20°C
					To 205° %	205° to 245° %	245° to 270° %	270° to 320° %	Residue above 320° %	Total %	Tar acids below 320°C C C	Sulphonation residue below 320°C C C	
1	Air Sec. No. 1 1875 Sound pile	5.10	13.2	unknown	none	20.4	11.3	26.7	41.6	100	8.5	1.2	2 semi solid—3 and 4 liquid (some crystal)
2	Wat Sec. No. 1 1875 Sound pile	6.12	13.2	unknown	none	23.3	18.6	21.2	36.4	99.5	5.0	1.4	2 semi solid—3 and 4 liquid (some crystal)
3	Mud Sec. No. 1 1875 Sound pile	9.25	13.2	unknown	1.1	36.7	14.4	19.7	27.7	99.6	10.0	1.1	1 and 2 semi solid—3 and 4 liquid (some crystal)
4	Air Sec. No. 2 1875 Badly attacked pile	6.49	14.3	unknown	none	16.8	13.0	20.4	49.8	100	8.2	1.1	2 semi solid—3 and 4 liquid (some crystal)
5	Mud Sec. No. 2 1875 Badly attacked pile	10.05	14.3	unknown	1.5	38.0	12.4	19.3	28.3	99.5	...	1.1	1 and 2 semi solid—3 and 4 liquid (some crystal)
6	Air Sec. No. 3 1895 Sound pile	9.71	15.1	24	2.2	55.5	8.2	12.3	21.6	99.8	10.3	0.8	1 and 2 solid—3 and 4 semi solid
7	Wat Sec. No. 3 1895 Sound pile	9.14	15.1	24	1.3	58.5	9.5	12.5	18.2	100	5.5	0.8	1 and 2 solid—3 and 4 semi solid
8	Mud Sec. No. 3 1895 Sound pile	10.60	15.1	24	3.3	56.6	9.7	12.1	17.8	99.5	11.7	0.8	1 and 2 solid—3 and 4 semi solid
9	Air Sec. No. 4 1895 Badly attacked pile	4.38	13.3	24	trace	48.4	10.1	13.7	27.8	100	9.1	0.7	2 solid—3 and 4 semi solid
10	Wat Sec. No. 4 1895 Attacked by lim	6.38	13.3	24	1.0	51.7	10.8	15.2	21.0	99.7	5.2	0.8	1 and 2 solid—3 and 4 semi solid
11	Mud Sec. No. 4 1895 Attacked by lim	9.28	13.3	24	3.2	57.9	9.8	10.8	18.2	99.9	11.8	0.8	1 and 2 solid—3 and 4 semi solid
12	Wat Sec. No. 5 1895 Attacked by teredo	4.86(a)	11.4	24	0.3	37.8	11.8	14.3	35.5	99.7	....	....	1 and 2 solid—3 and 4 semi solid
13	Wat Sec. No. 6 1895 Sound pile	8.83	22.5	24	(b)	49.9	8.8	13.6	27.5	99.8	....	....	2 solid—3 and 4 semi solid
14	Mud Sec. No. 6 1895 Sound pile	15.72	22.5	24	5.5	54.9	8.1	11.0	20.4	99.9	....	....	1 and 2 solid—3 and 4 semi solid
15	Wat Sec. 1895 Sound cedar pile	3.22	....	unknown	trace	44.1	23.9	17.5	14.1	99.6	6.1	....	2 chiefly solid (some liquid) 3 and 4 semi solid
16	Mud Sec. No. 7 1895 Sound pile	14.36	20.5	24	3.1	58.4	9.7	12.4	16.4	100	10.0	0.8	2 solid—1.3 and 4 semi solid
17	Mud Sec. No. 8 1895 Sound pile	14.16	20.2	24	4.6	62.2	8.8	10.5	13.8	99.9	13.6	0.7	1 and 2 solid—3 and 4 semi solid

(a) Estimated from water section assuming 40% evaporation.
(b) Through error, no observation made at 205°C.

of the oils takes place in the upper two sections which cannot take place in that section in the mud.

2. The small end of the pile, the upper end of the tree, being down, the percentage of the sap wood in relation to the sectional area of the stick is larger in the lower sections.

3. The occurrence of pitchpockets and pitch streaks in yellow pine, which greatly retard the absorption of oil, is more frequent in the upper ends of the piles, the butts of the trees.

4. It is a noteworthy fact that in a great many instances, when heavily creosoted piling are being pulled, oil drips from the piling points and it is an open question whether there is not some downward movement of oil after a great many years in an upright position.

5. Experiments have shown that in long-leaf yellow pine the ratio of the radial to the longitudinal penetration of the oil is about 1 to 25. All piling under consideration in this report being of short length, from 18 ft. to 26 ft., the percentage of the oil entering the stick through the end is very large and the analyses which are made on sections of stick which were taken a considerable distance from the end, would show approximately about 70% of the actual treatment received by the entire stock.

present in the mud sections of the two sample piles, are strikingly similar. A statement, that the oil at present in the mud sections of these piles is identical in character with the oil originally injected, might meet with some objection. It would seem, however, that as there could be no evaporation, in the section in the mud, it can be said without fear of contradiction, that the oil extracted from the mud sections of these piles, very closely approximates in character the oil originally used for the treatment.

Analyses 1 and 4 indicate the character of the oil taken from the air sections of the piles. A comparison of these with analyses 3 and 5 and a comparison of the amounts extracted in each case, indicate that in the 38 years between 30% and 40% has been lost from the air sections due to the evaporation of a part of the lighter constituents. A decrease is also noted in the percentage of "tar acids," which is somewhat greater than indicated by the analyses as more than 100 grams of the original oil is represented by the 100 grams used for analyses 1 and 4.

Analysis 2 shows character of oil at present in water section of Pile No. 1. Pile No. 2 being almost eaten away, no analysis was made of the water section of this pile. It is noted that there has been a disappearance of

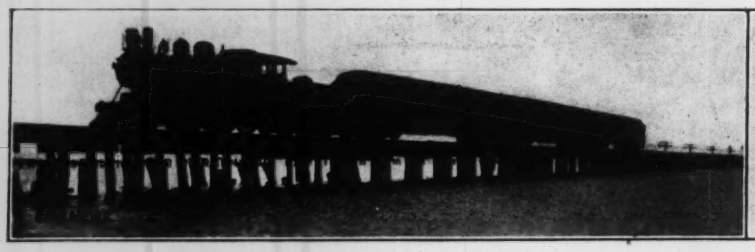


FIG. 3. SANTA FÉ RAILROAD TRESTLE ACROSS GALVESTON BAY PRIOR TO DISMANTLING IN 1913

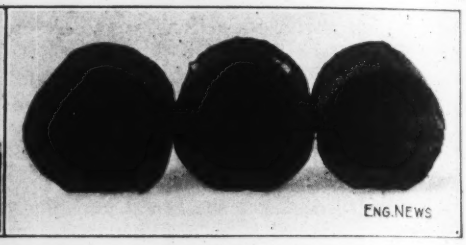


FIG. 4. AIR, WATER AND MUD SECTIONS CREOSOTED PILE NO. 3, TREATED 1895

Concerning the origin of the oil with which this oil piling was treated, little is known, except that it was an American oil and was at that time known as "light oil," the term *light* having no reference to weight or color but being used to designate a rather thin oil which would not solidify readily and used in contradistinction to the term "heavy" oil which denoted an oil containing a large percentage of soild constituents (rich in naphthalene) that could be liquified only at a rather high temperature.

At the time these piling were treated, creosoting plants and chemists were utter strangers to each other and no analyses of the oil were ever made, at least not on the part of the creosoting company. It is stated, however, that one crude test which was applied by the operator was to rub a little of the oil on the forearm and if it burned sharply it was a good oil, the burning sensation probably being due to the presence of a considerable amount of "tar acids."

Analyses 3 and 5, which are the analyses of the oil at

oil from this section also, due to evaporation, though the percentage of loss is somewhat lower than in the air section. A marked decrease of "tar acids," even greater than in the air sections, is noted. This seems to indicate that the disappearance of "tar acids" from creosote oil after injection into wood, while due in part to evaporation, is due chiefly to their partial solubility in water.

From the amount of oil found in the sections of the pile, it is estimated that the total actual treatment of Pile No. 1 is 13.2 lb. per cu.ft. and of Pile No. 2, 14.3 lb. per cu.ft. When it is considered that only about two-thirds of Pile No. 1 has been penetrated, the treated portion has, in reality, received a very heavy treatment. Being in the water, there was no checking and the top of pile not being cut off in such manner as to expose untreated wood, the heavily treated shell has preserved the pile.

The results of analyses of two of these 1875 piles made several years ago, published by Dr. Alleman, in 1908,

TABLE II

Analyses published by Dr. Alleman in 1908 in Forest Service Circular 98 "Quantity and Character of Creosote in Well Preserved Timbers" of two piles treated and put in the Santa Fe Trestle at Galveston in 1875.

Sample pile No.	Source	Species	Service yr.	Creosote per cu.ft. lb.	Distillation of extracted oil: To 205° %	205° to 245° %	245°C to 270°C %	270°C to 320°C %	320°C to 420°C %	Residue %	Total %	Solid naphthalene from distillates	Solid naphanthracene oil from distillates
50	Inter. Creo. & Constr. Co.....	Loblolly pine	29	17.63	1 26	27.60	22.43	31.22	12.02	5.13	99.68	25.41	48.15
51	Inter. Creo. & Constr. Co.....	Loblolly pine	29	17.08	2.18	31.06	18.21	36.04	8.17	4.13	99.79	28.16	41.12

in Forest Service Circular 98, "Quantity and Character of Creosote in Well Preserved Timbers," are given herewith.

The Trestle Built in 1895

As previously stated, the success with the 1875 creosoted piles was such, that in 1895, when the bridge was entirely rebuilt, it was decided to use creosoted piling. The trestle, as built in 1895, was about two miles in length and extended from the mainland across Galveston Bay to Galveston Island. Fig. 3 shows view of the Santa Fé bridge prior to its dismantling in 1912 and 1913.

In the distance are seen the old piling from one of the bridges destroyed at the time of the 1900 storm. The water at the deepest point is about 9 ft. at mean low water and runs down to very shallow water at the edges. The trestle contained 751 bents and was built with four piles to the bent. The total number and length of piles were as follows: 3, 30 ft.; 871, 26 ft.; 1013, 24 ft.; 695, 22 ft.; 445, 20 ft.; 80, 18 ft.; total, 3107 ft.

The piling were gotten in in good condition and none of them were shattered in driving. The number included 110 red cedar piles which were given a lighter injection of creosote oil. The remainder were strictly longleaf yellow pine, creosoted according to contract with 24 lb. of oil per cubic foot of wood. The piles would sink if allowed to fall in the water. This fact, the piledriver learned from experience after he had lost several sticks of piling.

All of the piling were furnished by Ricker-Lee & Co., of Galveston, now the International Creosoting & Construction Co., and were creosoted at their Galveston plant. This plant was subsequently moved to Beaumont, Tex., and is at present the Beaumont plant of the company. The pine piling were treated in the green state, the majority of them coming from the stump. They were seasoned by steaming in accordance with the following program. Steaming was regulated entirely by pressures with no attention given to temperatures. Only saturated steam was used in direct contact with the wood. Superheated steam was used in coils in the bottom of the treating retort during entire steaming and vacuum process. Total steaming process was of 36 hr. duration. The first 24 hr. the steam pressure was maintained at 55 lb. and for the remaining 12 hr. was kept between 47 and 50 lb. After steaming, a vacuum was maintained for four hours. After the vacuum process, the retort was filled with oil and same was forced into the timber under pressure until the 24 lb. per cu.ft. had been injected. The actual amount of injection was determined by the difference in gage readings taken before and after each treatment.

The above information with reference to the method of treatment is supplied by personal memoranda of R. L. Allardyce, now General Superintendent of the International Creosoting & Construction Co., and at that time the engineer in direct charge of treating operations for Ricker-Lee & Co. Unfortunately, the complete treating records were lost at the time of the Galveston storm. At the time this work was in progress, Mr. Bisbee was Chief Engineer of the Santa Fé but the work was more directly in charge of, partly, F. Merritt, now Chief Engineer of the G. C. & S. F. Ry. Co. and partly C. F. W. Felt, now Chief Engineer of the A., T. & S. F. Ry.

The piling were treated under a ten-year guarantee, it being agreed by the Creosoting company to replace free of charge any pile found defective at the end of that time. Not a single pile had to be replaced. The report of the official inspection made at that time is herewith reproduced.

July 5, 1905.

The examination of the piles treated in 1895 as shown by two piles that were pulled and thoroughly examined, that the damage done by teredo is superficial and has not affected the strength of the piles in the least, and I would say that as far as action of the teredo is concerned that none of the piling driven in 1895 have been weakened by this cause.

S. F. CLAPP.

At the time of the 1900 storm, the Steamer "Roma" was driven through the trestle, breaking off a number of the piling, a total of 270 piles having to be replaced on acount of the storm. This storm, also, so seriously damaged the Southern Pacific and the Galveston, Houston & Henderson trestles that they were abandoned and after that date, all rail traffic into Galveston went over the Santa Fé trestle. It has on this account been necessary from time to time to drive additional piling for reinforcement. The 270 piles put in after the 1900 storm as well as the later ones came from various sources and are not considered in this report.

The following compilation of figures will give some idea of the traffic to which this bridge has been subjected. Each engine moving over the bridge is figured as three cars.

Year		Number of cars
1900	Oct. to Dec.	83,572
1901		247,118
1902		245,422
1903		320,116
1904		293,454
1905		329,462
1906		349,942
1907		352,532
1908		392,808
1909		342,990
1910		353,427
1911		404,658
1912	To May 25	148,069

Fortunately for the identification of the 1895 piling, they were, according to specification, sharpened to a 4-in. square point and the work was very carefully done. They were the only piling in the bridge sharpened in this manner and the identification was easy and absolute.

The bulk of the material taken from the bridge was accumulated in the yard of the Santa Fé Tie & Lumber Preserving Co., at Somerville, Texas, but some of it was never shipped off the Galveston division of the railroad and was immediately reused on other work. When the dismantling was begun, a number of the piles were broken off, it being claimed by the engineer in charge that they could not be pulled on account of being driven very hard, the points being embedded in clay; but as they got away from the shore into deeper water, no difficulty was experienced in pulling the piles.

For the above reasons, it was, very unfortunately, impossible to get an actual count of all the good and defective 1895 piling taken out. It is estimated by Geo. E. Rex, General Manager of Treating Plants of the A., T. & S. F. Ry. System, that less than 10% of the 1895 piling pulled was unfit to be reused in other work. The writer believes Mr. Rex's estimate to be very conservative, having, in addition to going over the piling in the Somerville yard, spent some time on the trestle at the time the piles were pulled. On actual count of 65 of these piles as pulled, 58 were found to be in perfect condition, 6 slightly attacked by borers but good for further service and one that would be considered worthless. This lot of 65 was about 1000 ft. from the channel where con-

ditions are as severe as at any point on the bridge and it was stated at the time by Mr. Kleuver, in charge of the dismantling gang, that the worthless pile of this lot was the poorest 1895 pile that he had pulled up to that time.

Fig. 4 shows view of the air, water and mud sections of one of the 1895 piles (Pile No. 3). This pile shows a thorough penetration and is unattacked by borers. The analyses of these sections are shown in the table, Nos. 6, 7 and 8.

Fig. 5 is a view of air, water and mud sections of one of the 1895 piles (Pile No. 4), which showed considerable attack. It seems to be chiefly limnoria which has worked on this pile but there is some evidence of teredo attack also. The results of analyses of the sections are shown in the table, Nos. 9, 10 and 11. The pile is penetrated through but the absorption in the two upper sections is very much less than average.

photograph were not present when the pile was pulled. Freshly sawed surfaces were used for the photograph but the sections were cut off the pile several months previous. The penetration is through and through but as shown by both photographs and analyses is much more dense in the mud section.

Fig. 8 shows a stack of the good piling in the yard at Somerville, Tex. The pile in the foreground is No. 3.

It is noted with regard to the 1895 piles, that most of them were penetrated throughout by the creosote but in no instance, even where the amount of oil absorption is less than average, is there any sharp division line between the treated and untreated portions of the stick. This would indicate that the preliminary seasoning of these piles was more thorough than of those treated in 1875.

Mr. Merritt says, "We used about a thousand piles from the Galveston Bay Bridge at Brownwood, and all of

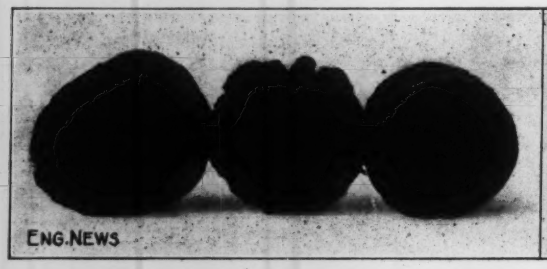


FIG. 5

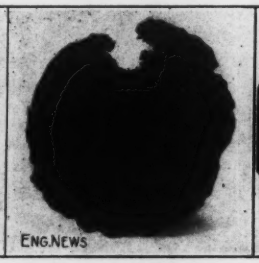


FIG. 6

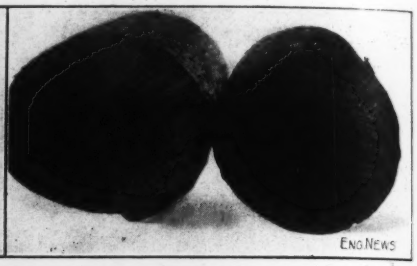


FIG. 7

Fig. 5. Air, water and mud sections creosoted pile No. 4 attacked by limnoria, treated 1895. Fig. 6. Section just below water line of pile No. 5 attacked by teredo, treated 1895. Fig. 7. Water and mud sections, pile No. 6, pile unattacked; treated 1895.

FIG. 8

FIG. 9

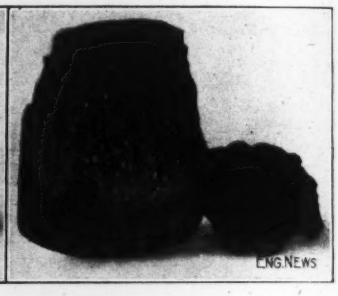


FIG. 12

Fig. 8. Stack of good creosoted piling treated in 1895 ready for reuse, in Santa Fé, T. & L. P. Co. yard at Somerville. Fig. 9. Sections just below water line of two creosoted red cedar piles, treated 1895. Fig. 12. Untreated yellow pine piles; limnoria and teredo action.

Where the 1895 piles have been attacked by borers, practically all of the attack is confined to a section not over 3 ft. in length just below the water line. They are damaged both by teredo and limnoria.

Fig. 6 shows an attacked section of one of the poorest of these piles (No. 5). This pile seems to have been attacked by the teredo alone. It is extremely resinous in character. The penetration of the oil is comparatively poor especially on the side of the attack. While 75% of the section is still untouched, the borers are through the creosoted portion and future action would have been very much more rapid. Analysis No. 12 of the table is the analysis of this pile. This is a very refractory piece of timber and has received a great deal less than an average amount of oil.

Fig. 7, analyses 13 and 14, represents water and mud sections of another of the 1895 piles (Pile No. 6) which is entirely free from any attack. The checks shown on them showed a remarkably good face after having been cut off to grade." This, however, includes other piles in addition to those treated in 1895.

With reference to the 110 red cedar piles which were treated, 53 of these were located in the yard at Somerville; 50 of them had been broken in pulling. Mr. Merritt states that some of these cedar piles were used at Brownwood and "were in most perfect condition."

Red cedar is a very difficult wood to treat, it being impossible to inject any large amount of oil into it. On account of the records being lost, it is not known just how much oil was injected, but it must, of necessity, have been a great deal less than was injected into the pine. In spite of this fact, it is found that the borers have attacked the cedar only very slightly. Not a single pile of those examined by the writer would have had to be discarded on account of attack by the marine borers.

Fig. 9 shows sections of two of the cedar piles cut just

below the water line. Analysis 15 is the analysis of the one on the right. It was untouched by borers but appeared to contain less oil than the one on the left which contained more checks and irregularities of contour.

The oil used in the treatment of the 1895 piles was what was then known as "heavy oil," not on account of its high specific gravity, but because at ordinary temperatures, it contained a large percentage of solid constituents. It was an important English oil brought over in barrels and it is stated that often the barrels would be more than half full of very hard, solid matter.

No analyses were made by the consumer and there is no record of any having been made by the manufacturer. To determine its character, we must refer to the oil extracted from the mud sections of the piles. It is found to be a low boiling oil very rich in solids, especially naphthalene. It contained about 60% of naphthalene and something over 12% of tar acids in the fractions distilling below 320° C. The small amounts distilling below 205° C. were solid, the fraction 205° C. to 245° C. was solid and the fraction 245° C. to 270° C. was semi-solid and contained naphthalene.

imum evaporation shown, is from the air section, and is approximately 25 or 30%. It is to be expected that piles showing the least absorption and poorest penetration would show the maximum amount of evaporation and that the location of the piles in the trestle, whether in the center or on the edge openly exposed to the sun, would also have an influence, but no record was kept of the location of the piles. A loss of "tar acids," which is most marked in the water sections, is noted in the case of both piles.

After the Galveston Causeway project was started, it was decided that some reinforcements were needed for the trestle and some untreated pine piles were put in. Fig. 10 shows one of these untreated piles, completely eaten in two by limnoria; there is also teredo action. Fig. 11 shows a stack of these piles discarded as worthless and shows how the limnoria has worked on them. Though recently put in, the limnoria has worked from the water line to the mud line.

Fig. 12 shows still another of the untreated piles. The sections are taken at the water line and just below it. The vertical section clearly shows the work of the lim-

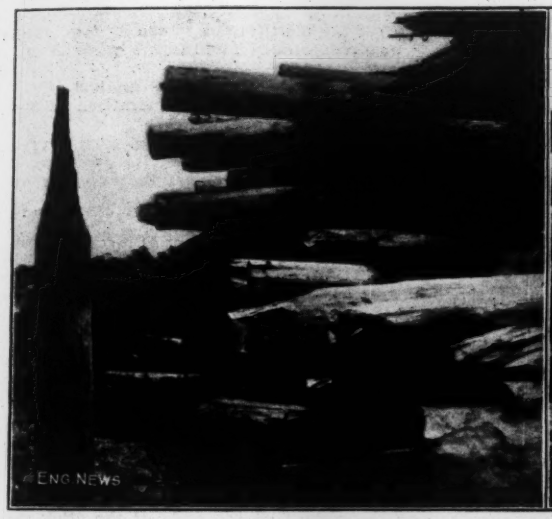


FIG. 10. UNTREATED YELLOW PINE PILES COMPLETELY EATEN IN TWO BY TEREDO AND LIMNORIA

FIG. 11. UNTREATED YELLOW PILES ATTACKED BY LIMNORIA AND TEREDO

The tar acids below 320° C. average 11.8 c.c. per 100 grams of oil. Figured to percentage by volume, this would be something over 12%, which is, even then, less than the total actual "tar acid" content of the oil as it does not include the "tar acids" present in the oil undistilled at 320° C. Small quantities of sulphonation residue were found on application of the sulphonation test, though from its general character this oil must certainly have been a pure coal-tar creosote.

A slight variation in the character of the oil extracted from the cedar pile, which is evidenced by a smaller amount of residue even in the water section, is accounted for by the fact that the cedar piles were treated last and near the end of the work some American oil was mixed in. The disappearance of oil from the upper two sections, due to evaporation, is, in the case of Pile No. 3, so slight as to be scarcely noticeable. In the case of Pile No. 4, which is one showing small absorption, the maximum evaporation shown, is from the air section, and is approximately 25 or 30%. It is to be expected that piles showing the least absorption and poorest penetration would show the maximum amount of evaporation and that the location of the piles in the trestle, whether in the center or on the edge openly exposed to the sun, would also have an influence, but no record was kept of the location of the piles. A loss of "tar acids," which is most marked in the water sections, is noted in the case of both piles.

noria while the cross-section shows that the teredo has also been at work. Over 75% of this pile has been eaten away.

In a great many instances, where creosoted piles have been attacked, it is noted that the borers have entered the wood through knots, and it is further noted in regard to the 1895 piles that most of the attacked sticks are very resinous in character. It is a fact well understood by operators treating yellow pine that knots and resinous pieces of timber are very refractory, that it is very difficult to force oil into them, and that there is a great deal of variation in the amount of oil absorbed by the various pieces in the same cylinder load.

From the examination of the 1895 piles and the analyses made, the writer believes that the sticks which show any considerable action by borers, are, in nearly every case, sticks which have received less than an average amount of oil due to their refractory character. A num-

ber of the untreated piles were examined with a view to determining if, in this case also, the knots were the first point of attack. Knots seemed to be very scarce but a few instances were found where the knots were less attacked than the softer wood surrounding them. This would lead to the belief that the cause of attack on knotty and resinous pieces of creosoted material is due primarily to the absence of creosote oil in a sufficiently large quantity and not to any particular fancy of the borers for this class of material.

To secure the best results in marine work, we believe that knotty and resinous piles should, as far as possible, be excluded by the specifications for the timber. The material should be given a full 24-lb. treatment of creosote oil and as much more as can be injected. In a number of specifications for creosoted piling for marine work, loblolly pine has been called for on account of its being softer and more easily impregnated with the preservative than long leaf; but it would seem that long leaf free from knots is much more easily obtainable than loblolly, and we believe this question to be one of very great importance in marine work.

We do not consider that we have sufficient data or that sufficient data are obtainable from the piling in the Galveston Bay Bridge to justify a conclusion that one particular grade of oil is better than any other, even for the limited field of marine work. The fact remains, however, that the 1895 piles, which have given excellent results, were treated with a high naphthalene oil and this fact should be accepted for what it is worth.

To summarize the results of the investigation we find as follows:

Out of an unknown number of yellow-pine piling creosoted in 1875, exact amount of injection (chiefly a light treatment) unknown and exact process of seasoning also unknown, after 38 years standing in Galveston Bay, some few are still in sound condition but the majority very badly attacked by limnoria and teredo and a great number are almost completely eaten away from mud line to water line.

About 3000 creosoted yellow-pine piles, strictly long leaf, creosoted in 1895, with a low-distilling creosote oil, containing approximately 60% of naphthalene and between 12 and 15% of "tar acids," an oil which would be excluded by practically all of the widely used present-day specifications in this country, seasoned by heavy steaming (probably by many considered excessive), injection of oil 24 lb. per cu.ft. of wood, have stood in Galveston Bay under heavy traffic for 18 years and over 90% still good.

For the benefit of those familiar with the retort method, the following distillation records made by the method of the American Railway Engineering Association are submitted. The oils are those extracted from the mud sections (approximately the oils used for treatment) of the 1875 and 1895 piles.

MUD SECTIONS 1895 piles 3 and 4		MUD SECTIONS 1875 piles 1 and 2	
To - 170°C -	None	To - 170°C -	None
170°C - 200°C -	1.3% semi-solid	170°C - 200°C -	0.5 semi solid
200°C - 210°C -	3.8% solid	200°C - 210°C -	1.1% semi solid
210°C - 235°C -	44.4% solid	210°C - 235°C -	21.4% semi solid
235°C - 270°C -	18.0% semi solid	235°C - 270°C -	26.2% semi solid
270°C - 315°C -	11.6% liquid (some solid)	270°C - 315°C -	17.6% liquid (some solid)
315°C - 355°C -	9.1% solid	315°C - 355°C -	14.3% solid
Residue	11.6%	Residue	18.9%
	99.8%		100.0%

The following is the analysis of the section above the water, of one of the 1895 piles as made by F. W. Kroemer, Chemist of the Santa Fé Tie & Lumber Preserving Co.

Amount of oil per cubic foot extracted	15 lb.
Tar acids by volume (distillate below 355° C.)	20%
Distillation of extracted oil (A. R. E. Assn. method)	
Up to 200° C.	2.6% liquid
200° C. to 210° C.	6.9% liquid
210° C. to 235° C.	29.2% liquid
235° C. to 270° C.	16.3% liquid
270° C. to 315° C.	13.3% liquid
315° C. to 355° C.	15.4% liquid
Residue	16.1% liquid

APPENDIX A—METHOD OF ANALYSIS

The samples of piling for analyses were placed in a turning lathe and a complete section of the stick turned off. The volume of the section of piling used was carefully measured. The oil was extracted from the turnings, using C. P. benzol in large glass Soxhlet extractors, by continuous extraction. After the extraction was complete, the mixture of benzol and creosote was filtered and the benzol distilled off through a Hempel column filled with glass beads to prevent the benzol carrying off any of the lighter constituents of the oil. After the benzol was driven off, the oil was weighed and from the volume of turnings used, was calculated to pounds per cubic foot. No attempt was made to separate the small amounts of resins that may have been extracted from the wood with the creosote oil.

The fractionations of the oil were made by the side neck flask method of distillation, outlet tube in middle of neck and thermometer bulb opposite the outlet tube. The distillates taken are those recommended by Dr. Alleman as giving the most valuable information in regard to a creosote oil, it being stated that in over 800 distillations which he conducted 92% of the oils which contained naphthalene gave it off between 205° C. and 245° C.

The tar acids were determined by the usual method of extraction with caustic-soda solution and neutralization of the extraction mixture with dilute sulphuric acid.

The sulphonation tests were made by using from 5 to 6 volumes of concentrated sulphuric acid at a temperature of 90° C. for several hours, with an occasional thorough agitation of the oil and acid mixture. The unsulphonated portions were separated with the aid of a centrifuge.

The estimated treatment of each stick is calculated from the amount of oil found in the mud section on the basis that with an average pile 22 ft. in length and 12 in. in diameter, a section of the pile any considerable distance from the end of the stick would show only about 70% of the average treatment for the entire stick. The writer realizes that it is rather hazardous to attempt to estimate the treatment of a pile from the amount of oil found in any section, and that the estimation is, to a certain extent, rather arbitrary. It is attempted in order to avoid the possible false assumption on the part of those unfamiliar with the treatment of timber, that the amount of oil shown by the analyses would represent the average treatment received by any pile.

It is well known to everyone familiar with creosoting operations, that a great deal of the oil entering a piece of timber goes through the ends, that the ends of the stick receive a much heavier treatment than portion of the stick away from the end and, that the shorter the stick and the larger the cross-section, the greater the percentage of the treatment which goes in the end. However, the actual percentage, if determined for one stick of timber, would not hold in another, even of like dimension, due to the variation in the closeness of the annual rings, the percentage of heart wood, the thoroughness of the seasoning, etc.

Recent experiments on long-leaf pine, by the United States Forest Service, have shown that for a maximum longitudinal penetration of 12 in. the ratio between the average longitudinal and the average radial penetration, varied from 32 to 1 to 22 to 1. It does not necessarily follow, that the amount of oil entering is in the same ratio as the distance penetrated, or even, that the same ratios would hold if the distances were materially increased, but it is certainly to be assumed that the amount of oil entering is in a ratio somewhat similar to the distance penetrated. A 22-ft. pile 12 in. in diameter has 1.57 sq.ft. of end surface exposed to longitudinal penetration and 69.12 sq.ft. of surface exposed to radial penetration, and if we should assume the above ratios we would find that from 42% to 33% enters the stick through the end. As part of the end surface exposed is heartwood, we believe, even in the case of these heavy treatments where both hardwood and sap wood are penetrated, that the actual amount of oil entering through the end is less than the above calculations would show. From the available data we conclude that with this particular class of material, sections of piles any considerable distance from the end would show approximately 70% of the average treatment of the entire stick.

Milwaukee Ave. Viaduct, Chicago*

SYNOPSIS—This viaduct crosses a group of railway tracks and is mainly of reinforced concrete. Special features of the design include very heavy girders of long span, a shallow floor with street-railway tracks, and very thin supporting bents or piers placed between the railway tracks. Special form design and special construction methods were necessary in order to avoid interference with railway traffic.

The Milwaukee Ave. and Desplaines St. viaducts, in Chicago, carry streets across a group of railway tracks and team yards between Kenzie and Wayman Sts., these facilities occupying a width of 460 ft. The two viaducts and three plate-girder steel spans on the south end of the Desplaines St. Viaduct, where the spans required were longer than could be made of concrete. The steel spans present no unusual features.

In designing the reinforced-concrete spans, it was necessary to meet the rather unusual requirement that these should carry the heaviest city traffic with a floor depth not much greater than in the old steel structure designed for the light traffic of 40 years ago. This was not difficult for the Desplaines St. Viaduct, as the city consented to a maximum rise of 8½ in. in the street grade. In the Milwaukee Ave. Viaduct, however, it was necessary to design through-girder spans, as the street

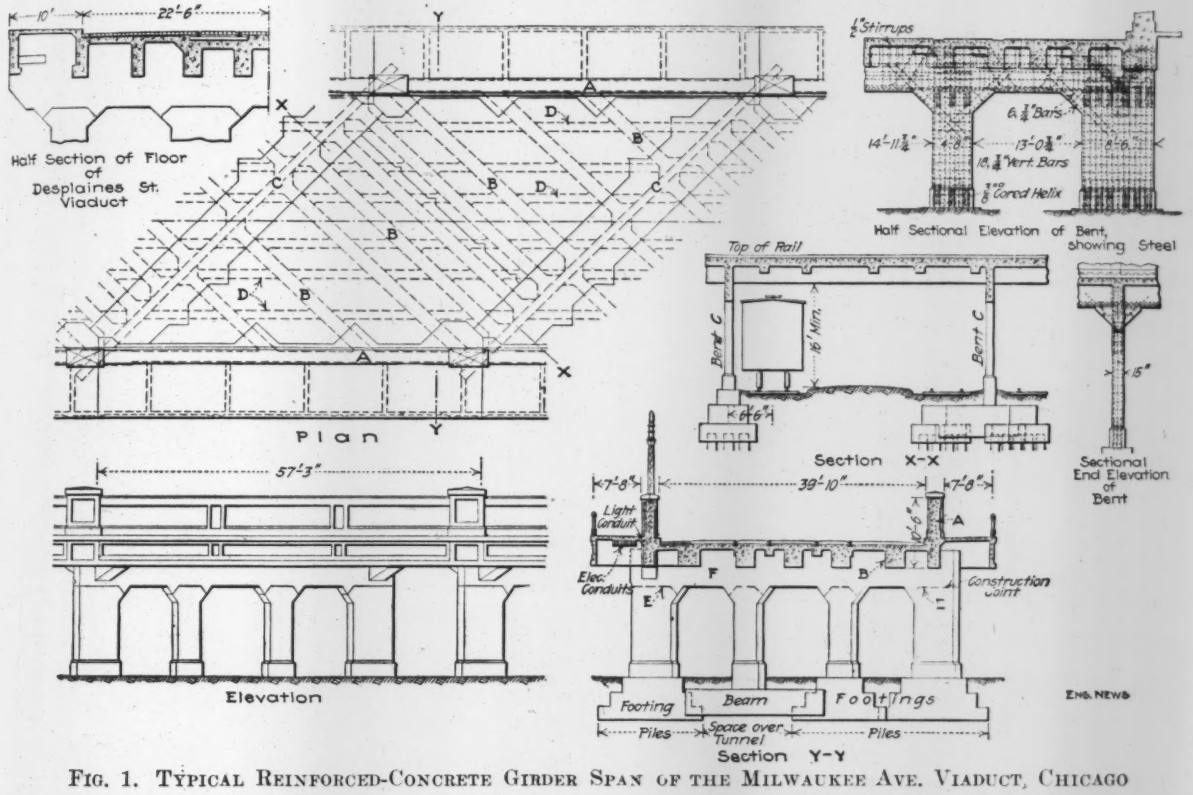


FIG. 1. TYPICAL REINFORCED-CONCRETE GIRDER SPAN OF THE MILWAUKEE AVE. VIADUCT, CHICAGO

form a V, the former being diagonal to and the latter at right angles to the tracks, with lengths of 524 ft. and 665 ft. respectively. The old viaducts were of iron, built in 1873, and were severely over-stressed during recent years. The new structures were designed under the city's specifications, the loading in which is about equivalent to Cooper's A-2 loading for highway bridges.

Preliminary estimates showed a reinforced-concrete structure to be cheaper than a steel structure with a concrete floor, besides requiring less maintenance. It was, therefore, decided to build eight reinforced-concrete spans on the north end of each structure, with a 156-ft. through-truss steel span and a 66-ft. plate-girder steel span on the south end of the Milwaukee Ave. Viaduct, grade and the headroom over the railway tracks limited the floor depth to 49 in. from the top of street-car rail to bottom of concrete. Fig. 1 shows a typical span. The main girders *A* have an average length of 55 ft., with a maximum of 57 ft. 11 in.

As the main girders are a little more than 42 ft. apart, and the average perpendicular distance between the bents is slightly less than this, the beams *B* were placed at right angles to the piers of bents *C* instead of at right angles to the girders *A*, as is more usual in through-girder construction. The four middle beams afford nearly all the support for the heavily loaded central portion of the roadway under the street-railway tracks. The arrangement of the floor system has the additional advantage of reducing the total load to be carried by the girder *A*, thus avoiding the inconveniently large dimensions which would otherwise occur.

*From information furnished by J. H. Prior. Formerly Engineer of Design, Chicago, Milwaukee & St. Paul Ry., Chicago, Ill.; now Assistant Chief Engineer Illinois Public Utilities Commission, Springfield, Ill.

By arranging the beams as shown and by the use of a special track construction it was possible to keep the floor within the required depth of 49 in. In the opinion of the writer, the design would have been complete and sufficient without the short stringers *D*, but these were inserted at the direction of the municipal authorities.

The main girders are shown in Fig. 2. They are 10 ft. 6 in. deep and 20 in. wide, each reinforced with 28 1-in. bars spaced 3 in. horizontally and 2 in. vertically. They also contain a considerable amount of vertical reinforcement necessary to provide for web stresses. The floor on the Desplaines St. Viaduct is similar in construction, but as it crosses the tracks at approximately 90° parallel to the center line of the girders to afford a sufficient bearing area along the bottom of the girders.

Bronze plates *A* and *B*, the upper surfaces of which were finished as a sliding surface, are fitted into the upper cover plates *C* of the grillage beams by means of a lug. Cast steel plates *D* and *E*, planed smooth and designed to slide upon the bronze plates *A* and *B*, were concreted into the under surface of the girder. It is believed that this detail will be a protection against excessive stresses due to expansion and contraction of the concrete structure, and that it will also prevent the formation of cracks around the bearings, which are sometimes to be found in concrete structures not provided with adequate expansion joints. A similar provision was made for expansion under each of the floor beams where they rest on the bents.

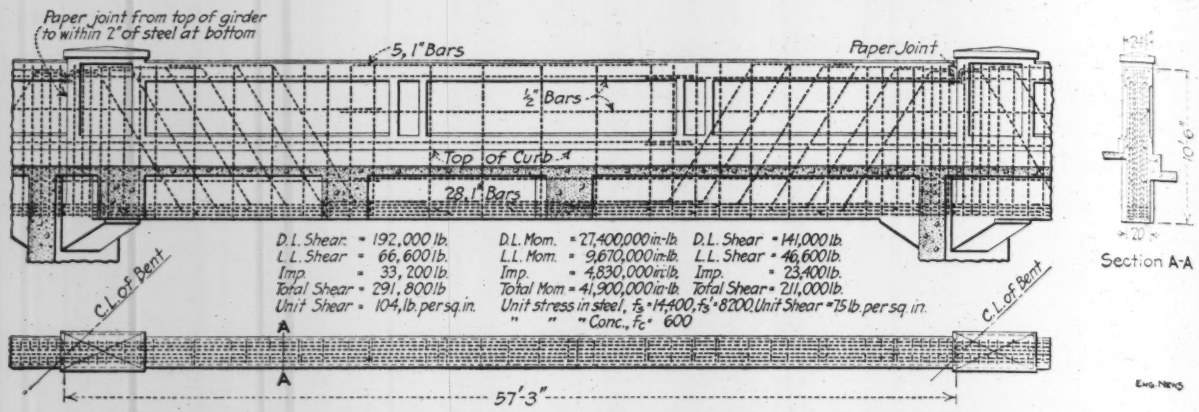

Paper joint from top of girder to within 2" of steel at bottom
5, 1" Bars
Paper Joint
½" Bars
Top of Curb
28, 1" Bars
D.L. Shear = 192,000 lb.
L.L. Shear = 66,600 lb.
Imp. = 33,200 lb.
Total Shear = 291,800 lb
Unit Shear = 104 lb. per sq. in.
D.L. Mom. = 27,400,000 in-lb.
L.L. Mom. = 9,670,000 in-lb.
Imp. = 4,830,000 in-lb.
Total Mom = 41,900,000 in-lb.
Unit stress in steel, f_s = 14,400, f_s' = 8200 Unit Shear = 75 lb. per sq. in.
" " " Conc., f_c = 600
C.L. of Bent
C.L. of Bent
A
A
57'-3"
Section A-A

FIG. 2. HEAVY REINFORCED-CONCRETE THROUGH GIRDER: 57-FT. 3-IN. SPAN, 10 FT. 6 IN. DEEP

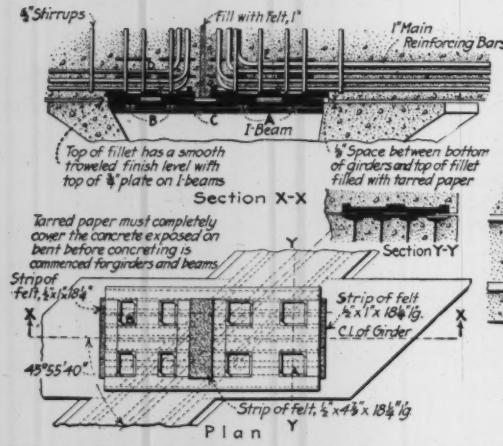

⅝" Stirrups
Fill with Felt, 1"
1" Main Reinforcing Bars
I-Beam
Top of fillet has a smooth troweled finish level with top of ⅜" plate on I-beams
½" Space between bottom of girders and top of fillet filled with tarred paper
Section X-X
Tarred paper must completely cover the concrete exposed on bent before concreting is commenced for girders and beams.
Section Y-Y
Strip of felt, ½ x 1" x 18¼"
Strip of felt ½ x 1" x 18¼" lg.
C.L. of Girder
43°55'40"
Strip of felt, ½" x 4½" x 18¼" lg.
Plan

FIG. 3. EXPANSION BEARING OF REINFORCED-CONCRETE GIRDER ON ITS SUPPORTING PIER

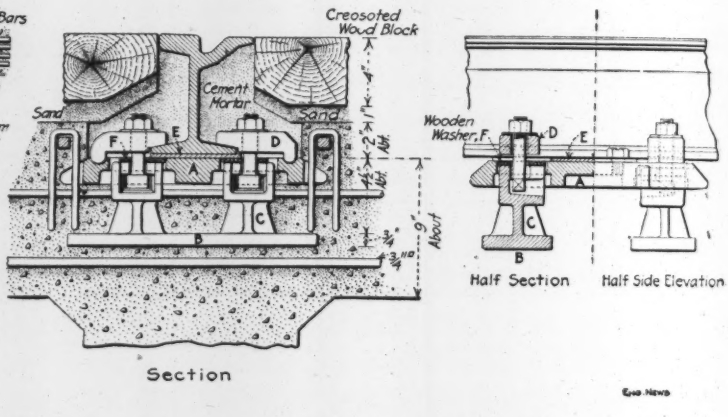


FIG. 4. TRACK CONSTRUCTION FOR SHALLOW FLOOR OF VIADUCT

there is no occasion for heavy girders. The concrete spans in both viaducts were arranged in groups of three, with fixed ends on the two center bents of each group and expansion ends on the two outside bents. The fixed ends of the beams *B* were built monolithic with the supporting bent, but at the expansion end they rest upon the bent and are tied together by a diaphragm, expansion being provided by means of a special sliding bearing.

In the Milwaukee Ave. Viaduct, the expansion ends of the main girders are carried on expansion bearings, Fig. 3. The lower half consists of an I-beam grillage, concreted into the bent. The lower beams of this grillage were made parallel to the bents to assist in distributing the load over the bent, and the upper beams were made

In order to maintain the former clearances for teaming purposes, it was necessary to place the bents midway between adjacent tracks, which were arranged in pairs. This limited the thickness of the bents to 15 in., necessitating the use in the columns of 1% of vertical reinforcement with hoops and spirals. Most of the columns are supported directly upon pile foundations but one column in each bent rests on a beam spanning the tunnel of the Illinois Tunnel Co., which runs directly beneath both viaducts about 50 ft. underground. Fig. 1 shows a typical bent carrying the fixed end of the girders. A con-

struction joint was made along the line *E-E*, the beam *F* being built as a monolith with the columns.

Special Track Construction

With the depth of floor and long spans required on Milwaukee Ave., it was not possible to use ties and ballast with the 9-in. grooved rail for the street-railway tracks, and a 7-in. rail with special chair support was used. This reduced the available floor depth from 49 in. to 42 in. only, while if ties and ballast had been used, the available depth would have been reduced an additional 7½ in., besides greatly increasing the dead-load weight of the floor. The chairs are 36 in. c. to c.

The rail chair, Fig. 4, was designed so as to interfere as little as possible with the beam action of the concrete and the reinforcement of slabs when placed in the structure, and at the same time to produce a suitable anchor a gap between it and the base-plate. This was remedied in the field by placing small wooden blocks between the outer ends of the anchor plate and bed-plate and omitting the wooden washers. On the Desplaines St. Viaduct, owing to the increased thickness of floor, the track was laid with ties and ballast (as in Fig. 1).

The trolley wires were strung overhead from reinforced-concrete trolley poles at each bent. On Milwaukee Ave., provisions for the lighting, power and telephone conduits were made by embedding 3½-in. fiber conduits in concrete in the space directly beneath the sidewalks. On Desplaines St., these conduits were placed in wrought-iron pipes supported on ledges on the stems of the sidewalk beams.

Construction Work

It was required that the new structures should be built

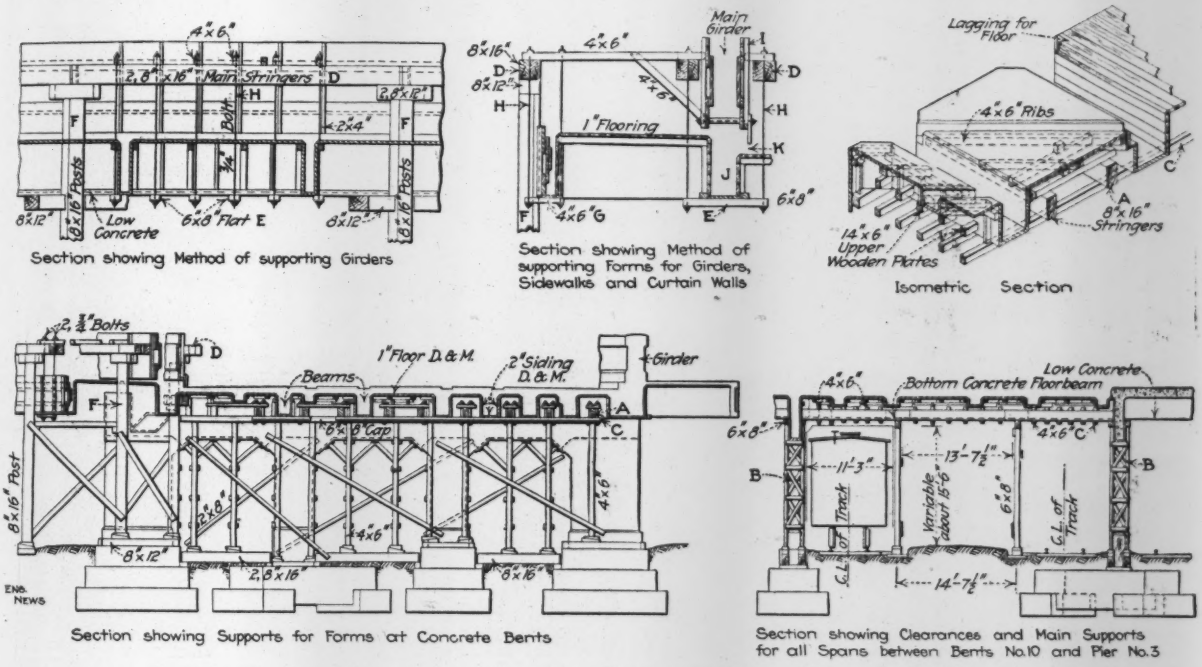


FIG. 5. FORM CONSTRUCTION FOR THE MILWAUKEE AVE. VIADUCT, CHICAGO

(The forms for the girders and spans had to be supported from above in order to avoid interference with traffic on the tracks beneath the viaduct.)

and sufficient bearing for the rails. This was accomplished by making the main part in two pieces. The bed-plate *A* is 16 in. square and 1½ in. thick, with four square holes for the sockets of the anchor plate *B*. Each of the two upright projections *C* terminates in a socket for receiving the head of a T-bolt. The anchor plate is placed in the floor between the upper and lower slab reinforcement, thus insuring a good anchorage when entirely surrounded with concrete. The rail is held in position by means of clamps *D* on the T-bolts. A 1¼-in. wearing plate *E* was provided between the bed-plate and the rail and secured to the bed-plate by tap bolts.

In construction, the chairs were clamped to the rail and the rail aligned and placed at its proper elevation before concreting. The ¼-in. wood washers were to act as temporary fillers and allow the anchor-plate to be drawn tight against the wearing plate. They did not prove satisfactory, as the tendency was to tighten the bolts too much, bending the wearing plate and causing practically without interference with the present railway tracks and without infringing on their lateral or vertical clearance. This was necessary as the tracks form part of a busy terminal. To meet this requirement the falsework shown in Fig. 5 was used on Milwaukee Ave. Fig. 6 shows the falsework and other features of the work under construction.

The distance from the lowest point of form work to the bottom of the permanent concrete floor was only 8 in., which unusually small projection of falsework below the concrete was obtained in the following manner. The 8x16-in. stringers *A* were placed between the stems of the T-beams *B*, which support the roadway floor. The ends of the stringers rest on falsework bents. To the underside of the stringers are bolted 4x6-in. pieces *C*, which support the bottom and side forms of the T-beams.

The forms for the main girders were supported as follows: 8x16-in. stringers *D* rest on the posts *F*, which pass through temporary openings in the concrete slab.

These stringers support the 6x8-in. pieces *E*, and the 4x6-in. pieces *G*, by ¾-in. bolts *H*. The pieces *E* support the bottom and sides of the lower portion of the forms for the main girder *G*. The forms *I* for that part of the main girder above the floor slab, are supported directly from the main stringers *D*, and are independent of the lower portion of the forms *J*. This arrangement permitted the main girders to be concreted practically as a monolith with the sidewalk and roadway slabs, by means of the openings *K*. The floor beams were also concreted with the main girders by maintaining similar openings at the ends of those beams.

The contractors were not allowed to interfere in any manner with the operation of trains on the main tracks or house tracks. On the team tracks, cars are spotted at night, and these tracks could be blocked between 7 a.m. and 6 p.m. An overhead clearance of 15½ ft. above top of rail was required.

In the construction of the Milwaukee Ave. Viaduct, the concrete in the footings (with the exception of the bent between the truss span and the three girder spans) was placed by a chute from a mixer mounted on a flat car. Materials were taken from cars on adjacent tracks and delivered to the mixer in wheelbarrows. For the superstructure, a tower about 50 ft. high was erected near the material tracks and a ½-yd. mixer on the ground discharged directly into an elevator bucket. The concrete was hoisted up the tower, dumped into a chute, and distributed by means of push carts of 6 cu.ft. capacity. A large part of the material was delivered to the mixer in wheelbarrows from cars on the material tracks, but a portion was brought in by teams and then taken to the mixer in wheelbarrows.

For work at the north abutment and the north spans, a mixer was placed on the north approach and the concrete discharged directly into place or distributed in push carts. In concreting the sidewalks and curbs on the steel spans, a ¼-yd. mixer mounted on wheels was used, and the concrete distributed in wheelbarrows. Materials were delivered to these mixers in wheelbarrows from storage piles in the street at each end of the viaduct. Near the south approach, a portable sawmill was erected for the form work. Electric power was used in operating the mixers, elevator and sawmill.

In the construction of the Desplaines St. Viaduct, the concrete for both the substructure and superstructure was placed by means of a tower and chute, the chute being long enough to deliver concrete to any portion of the structure.

The Milwaukee Ave. Viaduct has about 4000 cu.yd. of concrete and 600,000 lb. of reinforcing steel. The Desplaines St. Viaduct has 2100 cu.yd. of concrete and 315,000 lb. of steel. The concrete in general was 1:2:4 (using crushed limestone); in the columns it was 1:4, and in some of the footings 1:3:6. Tests of the concrete at 28 days, on cylinders 8 in. diameter and 16 in. long, showed 2500 lb. per sq.in. for 1:4, and 2000 lb. for 1:2:4 concrete. The cement gun was used for pointing after the forms were removed.

By agreement with the several railways, the work was designed by the Chicago, Milwaukee & St. Paul Ry.; C. F. Loweth, Chief Engineer, and J. H. Prior, Engineer

FIG. 6. CONSTRUCTION OF THE REINFORCED-CONCRETE SPANS OF THE MILWAUKEE AVE. VIADUCT, CHICAGO

(At the right is the elevator tower for the concrete, and the end of the steel truss span. Next to these are the forms for the concrete girders, suspended from overhead beams carried by bents between the tracks, the headroom being insufficient to permit of supporting the forms from below. At the left are the thin piers of one of the concrete bents, with the steel rods for the reinforcement of the top girder of the bent. This span of the viaduct crosses a freight platform and house tracks.)

of Design. The concrete work in the Milwaukee Ave. Viaduct was built by L. K. Sherman & Co., of Chicago, and in the Desplaines St. Viaduct by the Brennan Construction Co., Chicago. The steelwork on both viaducts was erected by the railway company's forces.

The field engineering, superintendence and inspection for the railway of both the steel and concrete were under the direction of R. J. Middleton, Engineer of Track Elevation for the Chicago, Milwaukee & St. Paul Railway.

Experiments on the Oxidation of Sewage at Columbus, Ohio

By C. B. Hoover* and C. D. McGuire†

SYNOPSIS—To throw light upon the value of sedimentation and oxidation of sewage, as practiced at Columbus, and to correlate the interpretation of laboratory tests with tests of samples from a stream receiving the effluent from sewage-treatment works, analyses extending over a series of years are reviewed. Dissolved oxygen studies indicate that proper observations of volume of stream flow in relation to available water for dilution, combined with observations of stream temperature and oxygen contents of both the stream and the sewage effluent may yet be made a guide for the operation of sewage-treatment works, but studies of the subject at various laboratories and comparisons of results are needed.

Within the last few years the most interesting and promising development on the laboratory side of sewage-treatment work has been the introduction and trial of the dissolved-oxygen-consumed test. The general purpose of this test is a study of the oxidation which takes place when sewage or sewage effluents are mixed in varying proportions with pure water, and the oxidizing action is measured by the dissolved oxygen content of the mixture before and after a definite period of incubation at a fairly constant temperature. It is thus evident that this method is an attempt to approximate the conditions which exist when a raw or modified sewage is discharged into a stream of water.

The problems which we wish to discuss may be more plainly set forth by a reference to Columbus conditions. The elements of the Columbus sewage problem are these: (1) a varying daily volume and concentration of sewage; (2) a varying daily river discharge; (3) a varying velocity of flow in the stream; (4) a varying daily air temperature. Years ago it became apparent that the sewage of the city and the water of the stream were not mixed in the proper proportion to prevent an intolerable nuisance from arising in the stream during the warm weather. It was impossible to increase the river discharge, consequently it became necessary to remove from the sewage a certain percentage of its nuisance-producing constituents. A sewage-treatment works consisting of sedimentation tanks, sprinkling filters, and final settling basins, was constructed to accomplish a modification of the sewage which would be sufficient to prevent a nuisance in the stream below the city. The modification is effected, broadly speaking, by sedimentation and oxidation.

The problems which confront us now are these: (1) How important is sedimentation in our scheme of treatment? (2) How important is the oxidizing device? (3) How can we give quantitative expression to the oxygen requirements of the sewage and sewage effluents? (4) What percentage of the oxygen-requiring constituents must be removed during the process of treatment?

These problems are of fundamental importance and must be worked out. They cannot be worked out in the laboratory alone because stream conditions cannot be established there, but it may be possible to establish certain relations between the results of actual stream tests and results which are secured in the laboratory so that laboratory results may be made to yield fairly reliable and practical data with respect to what conditions are apt to arise in a stream of a given discharge and oxygen content which receives a given volume of a sewage effluent which has a known oxygen requirement.

When we do not know the volume of our effluent, its oxygen requirement per unit of volume, and the oxidizing capacity of the stream at any stage and temperature, our work cannot be said to be on the most satisfactory basis. At Columbus, we have been trying to work out laboratory tests and methods of analysis which will reveal those qualities of the sewage and sewage effluents which are most important from a harmful stream-polluting point of view. Our work has been along the line of a development of the dissolved-oxygen-consumed test and particularly a study of the factors affecting the consumption of dissolved oxygen in mixtures of pure water and raw and modified sewages.*

Our method of conducting the dissolved-oxygen-consumed test is as follows: Take two clean grip glass-stoppered bottles of the same approximate capacity, which may be anywhere from 100 to 500 c.c., and pipette into each bottle enough of the sample so that when the bottle is filled with pure water, the desired dilution will have been provided. Fill the bottles with pure water by carefully pouring so that excessive aëration and air bubbles are eliminated. The water which is used for dilution is kept in the incubator long enough before being used so that its dissolved oxygen content becomes the same as that required to saturate water at the temperature of the incubator. This is done to prevent the incubated mixture from being supersaturated. After the bottles have been filled with pure water, one of them is sealed around the top of the neck with paraffin and placed in a 37° C. incubator, and the other is used for an immediate determination of the initial dissolved oxygen. At the end of 24 hours, the incubated bottle is removed and the residual dissolved oxygen determined. The difference between the

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*For an account of earlier studies of sewage treatment at Columbus see articles by Mr. Hoover on "A Method for Determining the Parts per Million of Dissolved Oxygen Consumed by Sewage and Sewage Effluents," and "Standard Tests of the Efficiency of Sewage Treatment," in "Eng. News" for Mar. 16, 1911, and Sept. 5, 1912, respectively.

initial and residual values multiplied by the dilution will give the oxygen requirement of the sample in parts per million.

The dilution which gives the most reliable result is one in which practically 50% of the initial dissolved oxygen is consumed during the period of incubation. We have found that for our crude sewage and septic tank effluent, a dilution of 32 (i.e., one part sample to 31 parts pure water) is about right and that for both the sprinkler and final basin effluents it is best to use dilutions of 4 and 8.

The results of the treatment of the sewage at the Columbus works when based on this test are summarized in Table I.

TABLE I. RESULTS OF DISSOLVED-OXYGEN-CONSUMED TESTS COLUMBUS SEWAGE WORKS, DECEMBER, 1909, TO JANUARY, 1914

	Crude sewage	Septic tank effluent	Sprinkling filter effluent
Total number of tests made	757	717	695
Dissolved oxygen consumed, parts per million			
Maximum value	376	288	66
Minimum value	4.3	3.5	3.0
Mean value	126	95	25

The mean values in Table I indicate that the reduction in the oxygen requirement by the treatment to which the sewage is subjected, is as follows: By the septic tanks, 24.6%; by the sprinkling filters, 73.7%.

Two pertinent questions naturally arise at this point and the burden of this paper is a consideration of these two questions, which are as follows:

(1) Is all of the oxidizable material in sewage and sewage-works effluents, oxidized in a period of 24 hours at blood heat when mixed in the proper proportion with pure well oxygenated water?

(2) Do the oxygen requirements of a sewage or sewage-works effluent, as determined by the laboratory test of dissolved oxygen consumed, express quantitatively the amount of oxygen which the stream must supply to meet these requirements?

In trying to answer question (1) we were naturally led into a study of the effect of the length of the period of incubation on the consumption of dissolved oxygen by mixtures of pure water and raw and modified sewages when incubated in the closed containers which we were using. The results of a large number of tests along this line showed that when a properly proportioned mixture of sewage and pure, well oxygenated water is incubated at blood heat, the consumption of dissolved oxygen will proceed for three days, after which it apparently ceases. There was no evidence in the results of these tests to show whether this cessation of activity was temporary or permanent, but the results did show, however, the rate at which oxidation takes place under these conditions and the rates of oxidation were found to be as shown by Table II.

TABLE II. PER CENT. OF THE TOTAL AMOUNT OF OXYGEN CONSUMED DURING THE FIRST THREE DAYS OF INCUBATION

	First day	Second day	Third day
Raw sewage	72	24	4
Septic tank effluent	74	19	7
Sprinkling filter effluent	72	22	6

We have now shown that the amounts of dissolved oxygen which mixtures of pure water and sewage or sewage effluents will consume during a 24-hr. period of incubation, at blood heat, do not represent the amounts of dissolved oxygen which such mixtures require for complete oxidation and furthermore that after three days of incubation the consumption of oxygen ceases either temporarily or permanently. The question of whether the three-day value represents the real absolute oxygen requirement of the sample we must leave unanswered at this time.

In trying to answer question (2), namely, as to the oxygen requirements of a sewage or sewage-works effluent, as determined by the laboratory test of dissolved oxygen consumed, express quantitatively the amount of dissolved oxygen which the stream must supply to meet these requirements? a comparison of stream and laboratory conditions was naturally suggested. The first point of striking dissimiliarity noted between laboratory and stream conditions was the fact that in our laboratory test the incubation is conducted in closed and sealed bottles, whereas, in a stream, the incubation takes place in an open channel in which the mixture of sewage and water has free access to the air. This difference in conditions suggested the desirability of comparing, under laboratory conditions, the incubation of mixtures of sewage and water in open and in closed containers. Accordingly, a large number of tests were made in which an incubation in closed and sealed bottles was compared with an incubation of a similar mixture in an open beaker. The results of these tests indicated that, under laboratory conditions, at least, there is a very marked difference in results secured between open and closed incubation. The closed incubation showed, roughly, an average dissolved-oxygen-consumed value which was 2.5 times as great as the average value for the open incubation. In explaining the apparently diminished oxidation in open containers, we can see but two alternatives: namely, diminished bacterial activity in open containers; or, undiminished bacterial activity and absorption of oxygen from the air. In view of the fact that temperature and dilution conditions were identical in each case, it seems reasonable to assume that there was no diminution in bacterial activity in the open containers and that absorption of oxygen from the air became an enormous factor in keeping the liquor well aërated at all times during the period of incubation.

We next investigated the effect of the extent of the exposed area in open containers on the consumption of dissolved oxygen. In making this test four different containers were used as follows:

(1) A closed and sealed glass-stoppered bottle of 160 c.c. capacity.

(2) Same as number one except that the glass stopper was not in place. The free surface thus exposed at the mouth of the bottle was 0.31 sq.in., from which the absorption of oxygen could take place.

(3) A wide mouth bottle, not stoppered, and with an exposed surface at the mouth of the bottle of 1.24 sq.in.

(4) A beaker with an exposed surface of 4.99 sq.in.

The average values, in parts per million of dissolved oxygen consumed, of a large number of tests of incubation in these containers, were as follows: No. 1, 79; No. 2, 66; No. 3, 54; No. 4, 14. These results show very conclusively that absorption of oxygen from the air increases with the extent of the exposed surface.

The next point which we investigated regarding incubation in open containers was the effect of the length of the period of incubation on the consumption of dissolved oxygen. The results of a large number of tests of the open incubation of mixtures of septic tank and sprinkling-filter effluents with pure water for periods of one, three, and five days, showed without exception that, when the proper dilution was used, the residual oxygen in such mixtures is lowest at the end of the first day, after which it

gradually and invariably increases. It has been shown elsewhere that in closed containers the residual oxygen in incubated mixtures of water and sewage decreases up to the third day, after which it remains practically stationary, and also that the rate of the consumption of dissolved oxygen is very rapid during the first 24 hr. and rapidly diminishes thereafter. In open containers the increase in residual oxygen after the first day seems to plainly indicate that absorption of oxygen from the air is not as rapid during the first day as the consumption of dissolved oxygen but that after the first day, absorption from the air is more rapid than the consumption of the dissolved oxygen.

The effect of the depth of liquor on the consumption of dissolved oxygen in open containers was next studied and the results of these tests showed that, in a short period of incubation and under laboratory conditions, there is no appreciable stratification with respect to bacterial activities where there is some depth of a mixture of water and sewage. In the Scioto River, which receives the effluent from the Columbus sewage works, even where there were sludge deposits, we have been unable, in a few tests that have been made, to find any difference in the dissolved oxygen of surface and subsurface samples.

The effect of the exclusion of light on the consumption of dissolved oxygen during the incubation of water and sewage in closed containers was next investigated and the results of this study showed that during a short period of incubation the presence or absence of light seems to be without effect upon the oxidizing activities of such mixtures.

The effect which an unlimited supply of air, from which oxygen might be absorbed, has upon oxidation in mixtures of sewage and water, has been previously shown. After these data were secured, it was decided to make similar tests where the supply of air was limited. The average values of five tests conducted at room temperature for 24 hr. are recorded in Table III.

TABLE III. SEWAGE OXIDATION TESTS WITH LIMITED AIR SUPPLY

Kind of container	Volume of mixture incubated in c.c.	Volume of air in c.c. from which oxygen could be absorbed	Dilution	Dissolved oxygen in parts per million initial	residual
Closed and sealed, 400 c.c.					
Bottle	400	0	2	6.88	0.04
Bottle	300	100	2	6.88	1.44
Bottle	200	200	2	6.88	2.52
Bottle	100	300	2	6.88	3.80

The effect of a limited air supply is quite marked in 24-hr. results and the question naturally arose as to what would happen during longer periods of incubation. This point was next investigated and the results are shown in Table IV, in which are recorded the residual-dissolved-oxygen values in parts per million in mixtures of equal parts of septic-tank effluent and pure water after an incubation at 37° C.

TABLE IV. STUDIES OF TIME ELEMENT IN SEWAGE OXIDATION WITH LIMITED AIR SUPPLY

	Conditions of incubation		
	Open beaker	300 c.c. bottles closed and sealed one-third full	two-thirds full
Initial dissolved oxygen in parts per million	6.2	6.2	6.2
Residual dissolved oxygen in parts per million			
After 1 day	3.4	2.4	0.6
2 days	3.8	3.2	0.8
3 days	5.0	3.6	1.2
4 days	5.8	4.4	2.4
5 days	6.2	4.8	1.4
6 days	12.4	4.8	2.2
7 days	14.4	6.4	2.2
8 days	13.6	22.8	3.2

As shown by Table IV, the residual oxygen in the open beaker with its unlimited air supply began to increase after the first day until the liquor showed supersaturation by the Winkler test—a condition which we have many times observed and which in this particular case was probably due to nitrites and was not, therefore, a true case of oxygen supersaturation. The residual oxygen in the closed and sealed bottle, which was only one-third full of the mixture, followed substantially the same course as that in the open beaker, which seems to indicate that there was a plentiful supply of oxygen in the 200 c.c. of air above the liquor. The residual oxygen in the closed and sealed bottle, which was two-thirds full of the mixture, followed a somewhat different course and the results seem to indicate that there was an insufficient volume of air present from which oxygen could be absorbed in amounts large enough to meet, first, the oxygen requirements of the liquor, and then to accomplish its saturation.

The next factor which was studied was that of dilution, and the results secured are not readily explained. The purpose of the test was to determine what effect dilution has on the consumption of dissolved oxygen at blood heat and in open containers. A similar sample in a closed container was also incubated at the same time. The results are as follows:

TABLE V. EFFECT OF DILUTION ON OXIDATION OF SEWAGE

Dilution	Initial dissolved oxygen in parts per million	Residual dissolved oxygen in parts per million: Closed and sealed bottle after one day	after two days	Open beaker after one day	after two days
2	6.0	0.0	0.0	3.8	5.8
4	6.4	0.8	0.4	4.4	6.4
8	6.6	1.4	1.8	5.2	6.2
16	6.6	3.8	1.8	6.2	6.4
32	6.8	5.4	3.8	6.6	6.4
Tap water blank	6.6	6.8	6.4	6.8	6.6

Table V shows how difficult it is to determine the oxygen requirements of sewage by incubation in open containers as, under these conditions, absorption of oxygen from the air apparently decreases as the dilution increases.

All of the tests heretofore reported have shown the results of oxidation during comparatively short periods of incubation. Table VI presents results of the incubation of mixtures of one part of tap water and one part of crude sewage, septic-tank effluent and sprinkling filter effluent at 37° C., for longer periods of time and in open containers. The volume of the mixture was in each case 3000 c.c. and the incubation was carried out in open glass battery jars.

TABLE VI. OXIDATION OF WATER AND OF RAW AND TREATED SEWAGE FOR 1 TO 13 DAYS

	Tap water blank	1 : 1 crude sewage	1 : 1 septic tank effluent	1 : 1 sprinkling filter effluent
Initial dissolved oxygen in parts per million	7.6	4.4	4.4	7.2
Residual dissolved oxygen in parts per million after 1 day	7.6	0.0	0.4	2.8
2 days	7.2	0.0	0.8	3.6
3 days	6.8	0.0	0.4	4.4
4 days	6.4	0.0	2.0	5.2
5 days	6.8	0.0	4.4	6.0
6 days	6.8	0.8	4.8	8.8
7 days	6.8	2.0	4.0	10.8
8 days	6.8	3.2	4.8	10.8
9 days	7.2	5.2	5.6	8.4
10 days	7.8	4.0	7.8	4.8
11 days	6.4	5.6	9.6	5.6
12 days	6.4	9.2	12.8	6.0
13 days	6.0	14.0	14.4	6.4

In the study summarized in Table VI the dissolved oxygen was determined by the Winkler method and the supersaturation found after varying periods of time created a doubt in our minds as to whether it was a true case of oxygen supersaturation. This study was then re-

peated using the regular Winkler, the acetate Winkler* and the ferrous sulphate methods of determining the dissolved oxygen each day for 13 days on samples of unfiltered and of filtered crude sewage to which two parts of pure water had been added.

Our experience with the three methods has been that they yield results that are in practical agreement except when nitrites are present in disturbing quantities, in which case the regular Winkler and ferrous-sulphate methods are inaccurate. It was noted in the mixture of unfiltered sewage and pure water that when there was no residual oxygen there was an offensive odor, but that just as soon as oxygen began to appear, the odor disappeared.

The question of whether the sewage solids are in an absolutely stable condition at the end of a thirteen-day period of incubation at blood heat must be left unanswered, but we are now investigating this phase of the problem and will have additional data to report along this line at a later date.

Practical Aspects and Summary

When raw or modified sewage is discharged into a stream of water, the practical question to the sewage-works manager is, what will the result be? Is it possible to answer this question from the laboratory? The condition of the stream will be the resultant of a number of factors, the most important of which are as follows:

(1) The volume of flow of the stream.
(2) The oxygen content of the water of the stream.
(3) The temperature of the water of the stream.
(4) The volume of the raw or modified sewage which is discharged into the stream.
(5) The available oxygen of the liquor which is discharged into the stream.
(6) The oxygen requirements of the liquor which is discharged into the stream.

All of the above factors are determinable. Assuming that the water of the stream is reasonably pure and practically saturated with oxygen, the problem which faces the sewage-works manager is that of adjusting his effluent to the dilution and temperature factors. When sewage is discharged into a stream of water there will be a critical period which will be the time during which the rate of the oxidation of sewage material will be very rapid, and after this critical period has been passed the oxidation will proceed at greatly diminished rates. The length of the critical period of time will decrease as the temperature increases. With sewage of a given strength and at a given dilution, the danger of producing offensive conditions will depend upon the disparity between the rate of the consumption of the dissolved oxygen of the mixed sewage and water and the rate of the absorption of oxygen from the air. The disparity between these rates will increase with the temperature and decrease with the dilution so that in order to prevent offensive conditions in the stream, higher temperatures should be accompanied by higher dilutions and lower temperatures will reduce the dilution requirements. The problem then is to get by the critical period, and, as the temperature and dilution factors are uncontrollable, it is up to the sewage-works manager to adjust his effluent to these factors whatever they may be.

Any improvement that is made in the character of the effluent would have the same effect in the stream as an increase in the dilution, or, in other words, it would make any given dilution more potent. The question now becomes, can you test an effluent in the laboratory and get results which will enable you to form an intelligent judgment as to what will happen when that effluent is discharged into a stream whose volume of flow, oxygen content, and temperature are known? We believe that by correlating river tests and laboratory tests along the lines which have been presented in this paper, it will be possible to work out such methods.

We are presenting these results in the hope that they may stimulate study along this line. Laboratory data are not as valuable in this line of work as they should be, but we are approaching the time when the laboratory will render an indispensable service.

*The acetate Winkler is the modification of the regular Winkler method which has been proposed by Hale and Melia in the "Journal of Industrial and Engineering Chemistry," Vol. 5, No. 12, p. 976, Dec., 1913.

Beach Erosion and Shore Protection, Atlantic City, N. J.

By Mandes Golder*

The coast of Atlantic City is formed of a sand deposit which is easily moved by the waves, from the action of which the shore line is constantly changing. The two views in Figs. 1 and 2 show how the beach has lowered at two particular pavilions since the footings for those pavilions were built. Inasmuch as a permanent, smooth beach is one of the requirements of a successful seaside resort, the separate interests which control the beach at Atlantic City have made it their practice to build shore-protection structures all along the frequented part of the beach, which is the entire city ocean front. Unfortunately, this construction has not been under the charge of any central authority, so that various methods have been used to prevent the beach from eroding. Some of these have been successful while others have not served the purpose for which they were designed.

At Atlantic City, the prevailing type of protection is the timber groin and timber wall, although in a few cases, stone groins have been built. The best wall which is now in service is the one illustrated in Fig. 3, which has been built by the Hotel Chalfonte owners to protect the beach in front of that hotel. It consists of a row of piles about 4-ft. c. to c. penetrating from 10 to 15 ft. and projecting about 5 ft. above the sand. Near the top of the piles, a 6x6-in. timber is fastened. Two rows of 2-in. sheathing are then driven in front of the 6x6-in. stick and fastened to it, and in front of the sheathing a riprap protection is dumped.

Another wall is built very nearly according to the same design as the one just mentioned, but the stone protection has been left off in front and instead there has been placed a banking of sand to the rear of the sheet piles.

Fig. 4 illustrates a groin built above Young's Pier It consists of 6x8-in. treated timbers, spaced 8-ft. center and 4 in. apart. Between these timbers are placed 4x12-in. treated plank, fastened to the timbers by galvanized-iron bolts. It is important in building this form of groin to get the plank about 3 ft. below the beach level so that there will not be any chance for water to wash under them. The plank between the centers are put in as the

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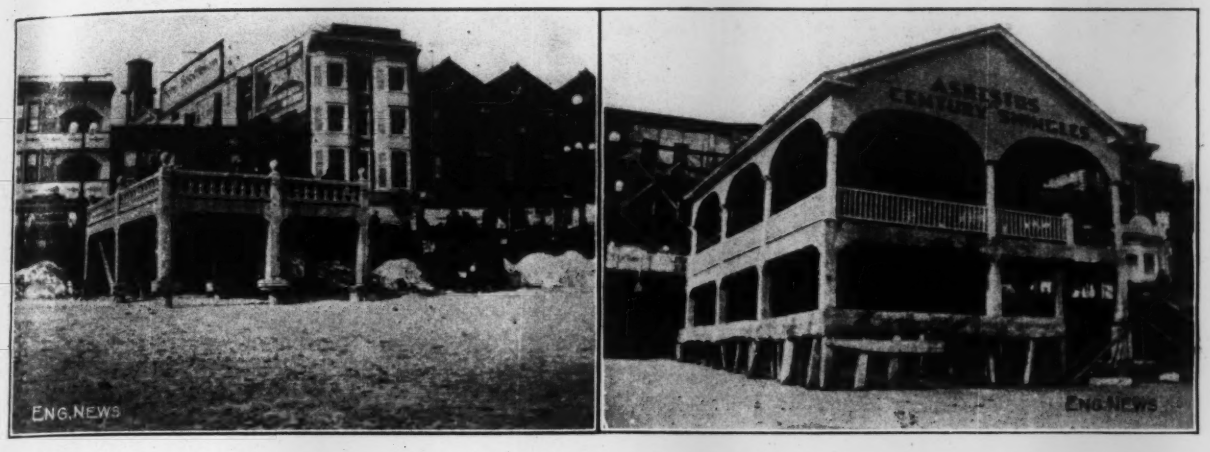
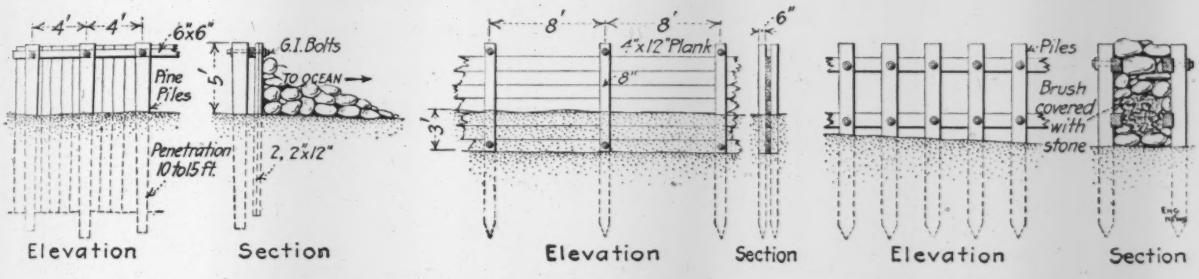


FIGS. 1-2. PAVILIONS AT ATLANTIC CITY, N. J., SHOWING PRESENT BEACH SEVERAL FEET BELOW FOOTINGS
(When built the footings shown were below beach level; the erosion shown is a matter of only a few years.)



FIGS. 3-5. SOME TIMBER GROINS FOR BEACH PROTECTION AT ATLANTIC CITY, N. J.

beach rises. The groin is built from the mean high-water line to the mean low-water line.

Another groin which has been used rather extensively is shown in Fig. 5. It consists of two rows of piles driven 2 to 3 ft. centers and about 4 ft. apart. Near the top, the piles are connected by a timber beam and the two rows are connected with bolts. In the space between the rows of piles, bundles of brush are placed and on top of that several layers of stone are laid.

All of the walls are built under the boardwalk, which runs along the beach front, so that they may be concealed from view.

In planning a system for protecting a shore and maintaining a beach in the State of New Jersey, there are three sources of information: (1) Surveys of the State and Federal departments; (2) information from residents in the vicinity or from the life-saving crews; (3) examination of defenses built as near the proposed work as possible. On the Jersey coast, the first source of information very often is quite valuable as surveys have been made in many places. Information from the second source shows the actual action of the waves during severe storms. From the third source, the effectiveness and weaknesses of a system already employed can be learned.

It is important to note that groins when well built and properly located, will raise the beach level but will not protect the shore line against storms. Shore protection, such as sea-walls, if well built, will protect the shore line but will at the same time cause a greater scouring action and thereby tend to lower the beach. Both forms of construction must, therefore, be combined in designing a beach-protection system.

Salaries of Engineers Employed by the State of Massachusetts—A Boston correspondent sends the following compilation of the annual salaries of engineering employees of the State of Massachusetts during the year ending July 1, 1913.

DIRECTORS OF THE PORT OF BOSTON—Chief Engineer, $6000; seven Assistant Engineers, $1800 to $3000; Designing Engineer, $2400; 28 Instrumentmen and Inspectors, $900 to $1800; Architectural Draftsman, $1800; three Designing Draftsmen, $1800; eight Draftsmen, $900 to $1500; 10 Rodmen, $2 to $2.50 per diem.

METROPOLITAN PARK COMMISSION—Chief Engineer, $4500; Assistant Engineer, $2400; four Transitmen, $900 to $1200; two Draftsmen, $1380 to $1500; Rodmen, $480 to $720.

METROPOLITAN WATER AND SEWERAGE BOARD—Chief Engineer, $6000; Division Engineer, $3600; Division Engineer, $2280; three Assistant Engineers, $1200 to $1740; Instrumentman, $1020; two Draftsmen, $1200 to $1500; Rodman, $600. Distribution Department, three Superintendents, $2280 to $2640. Sewerage Works, Engineer of Sewerage Works, $4000; Division Engineer, $1980; three Assistant Engineers, $1440 to $1800; two Draftsmen, $1200 to $1320; Transitman, $900; Rodman, $720.

MASSACHUSETTS HIGHWAY COMMISSION—Chief Engineer, $5000; First Assistant Engineer, $3000; Assistant Engineer, $2300; Chief Draftsman, $1800; four Division Engineers, $2500 to $3000; five Assistant Engineers, $1600 to $1800; 21 Assistant Engineers, $1200 to $1600; 37 Transitmen, $900 to $1100; 11 Draftsmen, $1000 to $1600; Rodman, $1 to $2 per diem.

MASSACHUSETTS STATE BOARD OF HEALTH—Chief Engineer, $5000; Assistant Engineer, $2400; four Assistant Engineers, $1260; two Assistant Engineers, $1140; three Assistants, $720 to $960.

Field and Office

A Guy Derrick with Interchangeable Parts and Collapsible Blocks

Several distinctive features are included in the Dunseath derrick, Fig. 1, just placed on the market. The foot-block and spider are interchangeable, as are also the boast seat and mast cap. The middle sections of the mast and boom are identical, so that the extension plates have the same dimensions. Holes are drilled at different points along the mast for connecting the lifting lug shown, for different drifts for lifting the mast from floor to floor. The blocks used are collapsible.

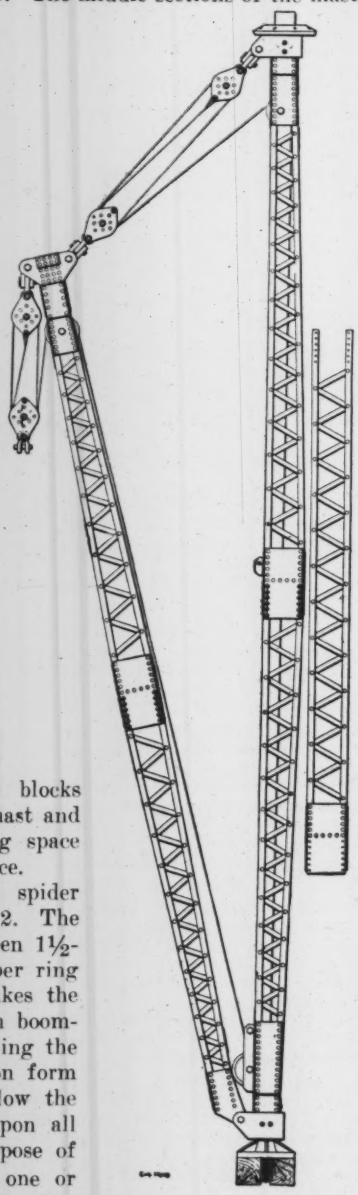


FIG. 1. DESIGN OF A 10-TON DUNSEATH DERRICK

Fig. 1 shows a 10-ton derrick. The foot and cap pieces, however, can be used with 10- to 50-ton equipment. As it stands, the mast is 50 ft. high and the boom 40 ft. long. When each is extended, the mast is 72 ft. and the boom 62 ft. With a boom whose length closely approaches that of the mast, the block shackles can be dispensed with, and the blocks bolted directly to the mast and boom caps, thus saving space and reducing interference.

The foot block and spider design is shown in Fig. 2. The load is carried by thirteen 1½-in. steel balls. The upper ring of 26 one-inch balls takes the kick-out of the boom in booming out. The bolts holding the seat and cap in position form loose connections to allow the load to adjust itself upon all the balls, with the purpose of preventing binding on one or two balls, with resulting trouble.

The details of the collapsible blocks are shown in Fig. 3. The blocks can be quickly built up for different numbers of sheaves; e.g., the one-sheave block shown at the right can be made into a three-sheave block by the addition of the necessary parts. The sheave pins shown are stepped. On long lifts the sheave making the most number of revolutions will, of course, turn on the largest diameter of the pin. Ordinarily, the diameter of the pins will be the same for all sheaves of a block. Where the plates fit around the pins, ring slots about ¼ in. deep are cut in the pin. The sheave-pin holes in the plates are a trifle larger than the largest diameter of the pin and tapered at one side to fit over the smaller diameter.

The sheaves are of openhearth cast steel, the pin of cold-rolled steel and the shackles of annealed cast steel.

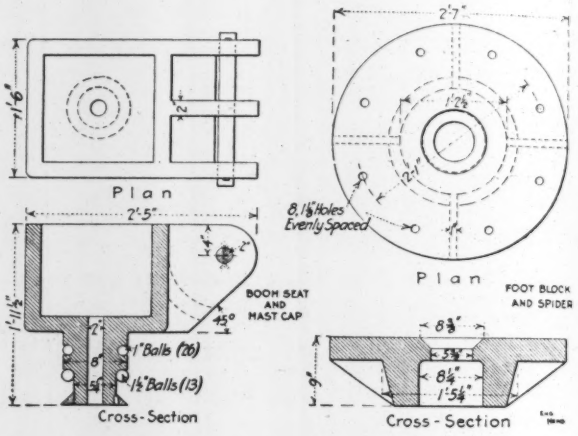


FIG. 2. DESIGN OF THE INTERCHANGEABLE SEAT AND CAP, FOOT BLOCK AND SPIDER

When the tapered pin is used, the sheaves are provided with phosphor-bronze bushings. The bushings are of varying diameters but will fit in any sheave.

The load shackle pin is locked by a key and pin. Fig. 4 shows a four- and a three-sheave block connected together. A complete 60-ton block, equipped with four 14-in. sheaves, weighs 360 lb.

In lifting the derrick from floor to floor, the boom is first drawn up against the mast and four guys are attached to its top. The boom is then picked out of its socket and the butt laid on a block of wood alongside. The boom guys are then run out and fastened. A rope goes over an auxiliary sheave on the same pin as the load sheave on the boom, down to a block on the boom footing and up to the load pulley, where it is fastened. The other end of the auxiliary rope runs to the lifting lug on the mast. A pull now lifts the mast which, when in position above, is secured, and then proceeds to pick up and set the boom by means of the boom falls.

Robert Dunseath designed these derricks and blocks, which are rented or sold by the Collapsible Block Co., 50 Church St., New York City.

Proper Care of Road Machinery*

Road machinery is an expensive investment, but doubly so if care is not exercised in its maintenance. If a few simple and inexpensive precautions are observed, it will result in an annual saving of thousands of dollars to the counties.

In the case of rollers, rock crushers and crusher power, which, during operation, are constantly exposed to dust, a liberal supply of oil or grease should be used to flush all bearings and thus prevent the rapid wear resulting from the presence of grit and foreign matter in dry bearings. Good lubrication of all machinery is the first step toward its economical maintenance.

Keep a good supply of extra parts on hand in case of breakage, especially for rock crushers. An extra link for the elevator chain may save many valuable hours. Extra crusher dies should always be on hand.

A tool box equipped with waterproof cover and fastened with a padlock is a necessity. This box should contain, among other things, a good supply of bolts of assorted sizes and lengths, with washers to match; also rivets, screws, nails and spikes. In brief, keep constantly on hand a generous supply of everything likely to be needed in case of a breakdown.

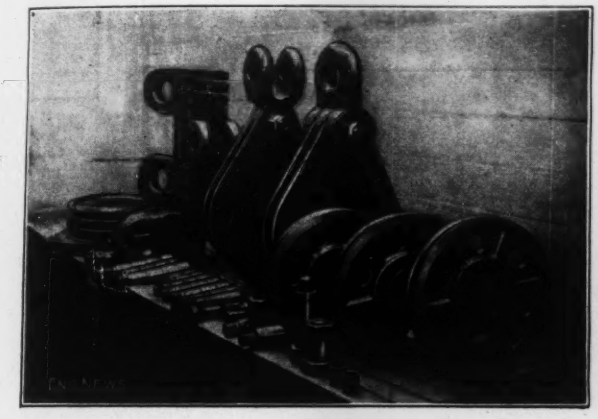


FIG. 3. COLLAPSIBLE-BLOCK PARTS

Insist that the roller operator keep his machine clean and in good repair. All grease and dust should be removed from the machine some time before the close of each day's work. A roller that has the appearance of a piece of junk after a single season's work is evidence of inexcusable neglect.

On the larger jobs, a small portable blacksmith's outfit is very useful. With this equipment, rooter and plow points and small tools can be sharpened as often as may be necessary without the delay incident to sending them to a blacksmith. Dull tools make expensive work.

When the season's work is completed, all machinery should be cleaned thoroughly, and a coating of heavy oil applied to all polished surfaces, boiler jackets, shafts, etc., thus preventing rust. Also paint smoke boxes and stacks. Machinery should be stored under cover. It is not necessary that storage sheds have tight sides, but a good roof is essential. It is not the usual practice to keep rock crushers under cover, but if this is done the machine will be in better running order when work is again begun.

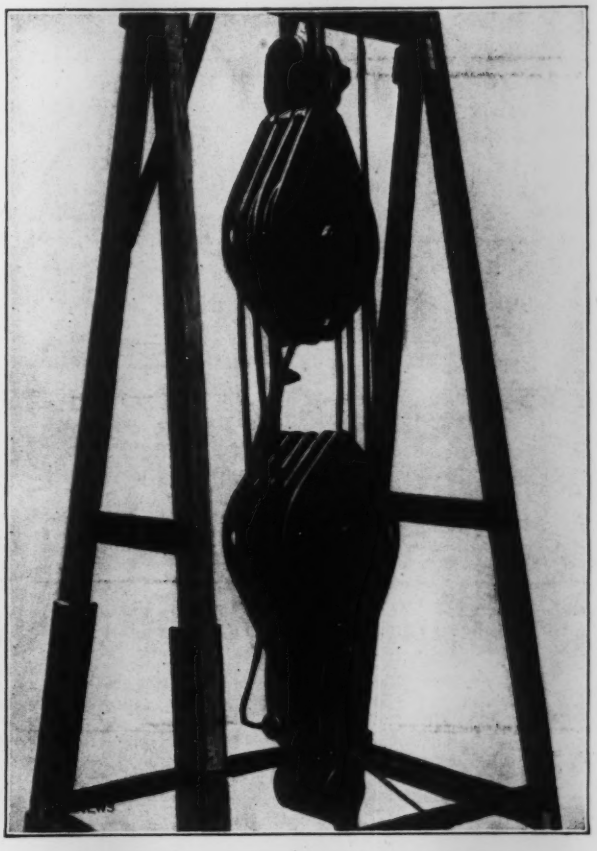


FIG. 4. COLLAPSIBLE BLOCKS ASSEMBLED

Rollers, road machines, power engines, wheelers, slips, plows and sprinkling wagons should always be protected during the winter season. Drain all water from all machinery before leaving it.

*From the Wisconsin Highway Commission's "Instructions to County Commissioners and Foremen for Building State Aid Roads," compiled by A. R. Hirst, State Highway Engineer, Madison, Wis.

Accurate Triangulation with a 5-In. Engineers' Transit

BY MELVIN E. LUTZ*

Most textbooks on surveying and geodesy give but scant mention of the ordinary engineers' transit as an instrument for tertiary triangulation. Some results which I have obtained with such a transit may be of interest.

I recently found it necessary to extend triangulation along what is known as the Hamakua coast of the Island of Hawaii. The general character of the country is a narrow cane belt 2 to 3 miles wide fringing the shore line and sloping toward the high interior on about a 12% slope. The interior is covered with a dense forest, and the cane belt is intersected by numerous deep gulches. The country traversed can properly be classed as rugged, and difficult to triangulate on account of the convex coast line.

*Assistant Government Surveyor, Honolulu, T. H.

SURVEY COST DATA COMPILED BY FRANKLIN & CO., CIVIL ENGINEERS, PHILADELPHIA, PENN.

Character	Location	Size	Unit day in hr.	Time Field	Time Office (one man)	Cost Field	Cost Office	Total Cost	Unit cost	Remarks
Topography	Pennsylvania	308 acres	6¼	40 days 3 men	10 days	$327	$45	$372	$1.20 per acre	95% = points established on 5-ft. contours with Y-level, and located with plane-table. 5% = points established on 5-ft. contours by, and located with stadia traverse = (woods). Land hilly with rise of 120 ft. Survey made Aug. to Oct. Scale of map—1 in. = 100 ft.
Topography	Pennsylvania	190 acres	7	22 days 3 men	9 days	$194	$38	$232	$1.22 per acre	80% = points established on 5-ft. contours with Y-level, and located with plane-table. 40% = points established on 2½-ft. contours with Y-level, and located with plane-table. 20% = woods = points on 5-ft. contours established and located with stadia traverse. Land hilly with rise of 180 ft. Survey made in Dec. and Jan. Scale of map—60 in. = 1 ft.
Topography	Pennsylvania	104 acres	7½	8 days 3 men	1 day	$74	$4	$78	$0.75 per acre	70% = points established on 5-ft. contours with Y-level, and located with plane-table. 30% = woods = points on 5-ft. contours established and located with stadia traverse. Land = rolling = rise of 80 ft. Finished plan not made. Survey made in June. Scale of map—1 in. = 100 ft.
Topography	Pennsylvania	300 acres	8	40 days 3 men	22 days	$430 including subsistence	$92	$572	$1.90 per acre	70% = points established on 6-ft. contours with Y-level, and located with plane-table. 40% = points established on 3-ft. contours with Y-level, and located with plane-table. 30% = woods = elevations obtained by stadia and vertical angles-contours interpolated. Extra large number of buildings, railroads, etc., were located. Scale—1 in. = 100 ft. Land = rough and mountainous = rise of 430 ft. Survey made Nov. and Dec.
Topography	Pennsylvania	99 acres	7½	12 days 3 men	4½ days	$115	$23	$138	$1.20 per acre	Points established on 5-ft. contours with Y-level, and located with plane-table. Land = rolling = rise of 60 ft. Survey made in Feb. and March. Scale of map—1 in. = 100 ft.
Topography	Pennsylvania	1464 acres	8½	87 days 3 men	18 days	$1052	$92	$1144	$0.78 per acre	90% = points established on 10-ft. contours with Y-level, and located with plane-table. 10% = woods = points on 10-ft. contours established and located with stadia traverse. All roads traversed and chained. Land = hilly = rise of 150 ft. Survey made Sept. to Dec. Scale of map—1 in. = 200 ft.
Topography	West Virginia	11,264 acres (259 mi. of stadia)	8	222 days 4 men	65 days	$3791 including subsistence	$386	$4177	$0.37 per acre (Topography) $14.63 per mi. (stadia lines)	Survey made entirely with stadia. Tape never used. Rough and mountainous, streams, ridges, and 2 coal outcrop lines traversed and topography taken by stadia and vertical angles. Contours interpolated. Levels on transit lines carried along by stadia and vertical angles. Survey made Oct. to Feb. Rise in elev. = 1150 ft. Stadia lines tied on to outline. Survey made 3 months previous. Scale—1 in. = 500 ft.
Railroad preliminary and estimate	No Carolina	26 mi.	8½	74 days 4 men	24 days	$1450 including subsistence	$144	$1594	$61.30 per mi.	Transit line chained. Topography taken with Y-level and cross-sections. Land = hilly and wooded. Projected line = 21 miles. Survey made June to Aug. Scale of map—1 in. = 200 ft.
Railroad preliminary and estimate	Virginia	27.5 mi.	8½	75 days 4 men	25 days	$1400 including subsistence	$150	$1550	$56.40 per mi.	Transit line chained. Topography taken with Y-level and cross-sections. Land = 50% woods and mountainous and 50% open. Projected line = 27½ mi. Survey made Aug. to Nov. Scale of map—1 in. = 200 ft.
Railroad preliminary and estimate	Pennsylvania	12 mi.	8½	50 days 4 men	35 days	$1011 including subsistence	$165	$1176	$98 per mi.	Transit line chained. Extra large number of locations of towns, railroads, etc., were made. Land = rolling and open. Projected line = 9 mi. Topography with transit (stadia and vertical angles) and plane-table. Made in Feb. and Mar. Scale of map—1 in. = 200 ft.
Railroad preliminary and estimate	Pennsylvania	5.1 mi.	8½	14 days 4 men	14 days	$236 including subsistence	$43	$279	$54.60 per mi.	Transit line chained. Large number of extra locations were made. Topography with transit (stadia and vertical angles) and plane-table. Land = rolling and open. Projected line = 3.6 mi. Made in May-June. Scale of map—1 in. = 200 ft.
Railroad preliminary and estimate	Cuba	34 mi.	8	43 days 9 men (inc. axemen)	51 days	$848	$238	$1086	$32 per mi.	Transit line partly chained and partly stadia. Scale of map—1 in. = 200 ft. Topography with Y-level and transit (stadia and vertical angles) Land = mountainous and heavily wooded. Survey made Nov. to Jan. No maps of country. Cost of reconnaissance therefore was greatly increased.

Note: Field cost includes transportation (except to Cuba). Field cost includes subsistence only where noted.

Complete observations were made on 26 triangles. The shortest line between stations was 900 m. in length, the longest line was 4950 m., and the average length of line was 43[illegible] m. The average error of closure of the 26 triangles was 3.6″, and the maximum error of closure was 12.4″.

The observing program was as follows: either two or three sets of observations were made at each station, the number depending upon the agreement of the first two sets. Each set consisted of six repetitions on the angle direct, followed immediately by six reverse on the explement.

The instrument used was a 5-in. Berger transit, which was protected from the direct rays of the sun by an umbrella. The instrument had been in continuous use for the year previous, but had always received good care in handling. The verniers read to single minutes, and with the aid of a good simple microscope I estimated to 10″.

The foregoing results seem to indicate that a good engineers' transit in good condition will do very well for tertiary triangulation when a better and larger instrument is not available, as was my case; or when the country is rugged and lightness of instrument is essential.

Topographic Survey Cost Data*

The accompanying table gives cost data for a number of different surveys, chiefly in Pennsylvania. Under the column "Remarks" is briefly stated the character of the country, the method of doing work, the time of year, and any notes which affect the cost.

In taking the Pennsylvania topography each 5-ft. contour was traversed by the rodman along points established by the Y-level and located by plane-table. The excessive cost of the fourth item was due to the extra large number of railroad tracks, buildings, etc., that were located.

The topography in West Virginia was taken by running stadia and vertical-angle transit lines along all streams, ridges and two coal outcrop lines, one of which was about one-third and the other two-thirds of the distance up the mountainside. Elevations were taken from all these lines (which formed a gridiron system) and the 50-ft. contour lines were interpolated.

The railroad preliminaries (with the exception of the Cuban work) were all tape-measured transit lines, with topography taken either with cross-sections, stadia and vertical angles or the plane-table.

In Cuba, the transit line was partly measured with tape and partly by stadia and vertical angles. No maps of the country could be found so that the reconnaissance was necessarily much more extensive than usual, thus increasing the cost. The owner provided subsistence of the men, which amounted to about $18 per mile.

*Compiled by Franklin & Co., Civil Engineers, 1420 Chestnut St., Philadelphia, Penn.

A Traveling Tamping Machine for Earth Fills

Well compacted earth is essential in trench filling and similar work, and for efficiency and economy, mechanical tampers should replace hand work. A portable mechanical tamper recently developed, which strikes about 50 blows per min. with a 150-lb. tamper, and a 22-in. stroke, offers many points of interest to engineers and contractors.

This machine is essentially a light, self-propelling drop-hammer, straddling on wide-gage trucks the ditch or zone to be tamped. Power for raising the tamper and for traction is furnished by a 4-hp. vertical water-cooled gasoline engine mounted over the forward axle. The tamping frame can be moved transversely to cover any ordinary width of ditch. The depth at which the tamper will work is determined by the length of the ram timber

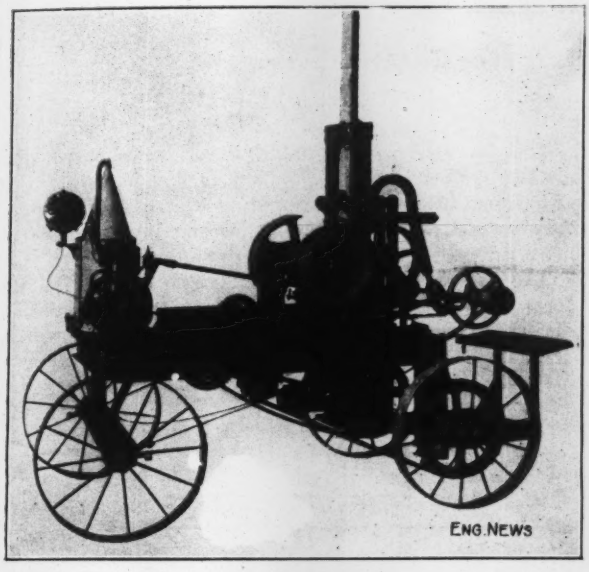


FIG. 1. A TRAVELING TAMPING MACHINE FOR COMPACTING EARTH FILLS

used, and the equipment is designed to take care of trenches of any ordinary depth. The entire operation of the machine, traction and tamping, is under the control of an operator seated at the rear. The sale price of the machine complete is $450.

The tamping is done by a hardwood 2x6-in. board, reinforced on its edges by steel straps, and carrying at its lower end a cast-iron tamping head 12 in. wide. This vertical ram is intermittently clamped between a segmental wheel and an idler pulley and is lifted a height equal to the length of the arc of the segmental wheel. The segmental drive wheel is mounted upon a splined shaft, driven through a countershaft from the engine. The idler wheel is swung in a pivoted frame. Pressure between the segmental wheels and the idler pulley against the board is obtained by means of a spring.

The frame in which the tamping ram is mounted has a transverse movement of 20 in., and the net width of the tamping zone is 32 in. The frame carrying the tamping ram and lifting mechanism can be moved transversely by an arrangement of light cable wound on drums, and a hand wheel controlled by the operators.

For raising the tamper when moving, etc., a lever throws the idler wheel out of contact with the beam, and locks it in place with a pair of toggle links.

The machine is self-propelling, with a road speed of about 1½ mi. per hr., and a working speed when tamping, of from 10 ft. to 25 ft. per min. Gearing is provided for reversing both the road and the working speeds. The wheel gage has a 2-ft. adjustment from 4 ft. 8 in. to 6 ft. 8 in., and the rear wheels are so mounted as to permit of a 10-in. vertical adjustment. With this adjustment, the machine is supported horizontally with the back wheels traveling at different elevations such as are often found when working along one side of a crowned road, on partly paved work, across an incline, etc. The

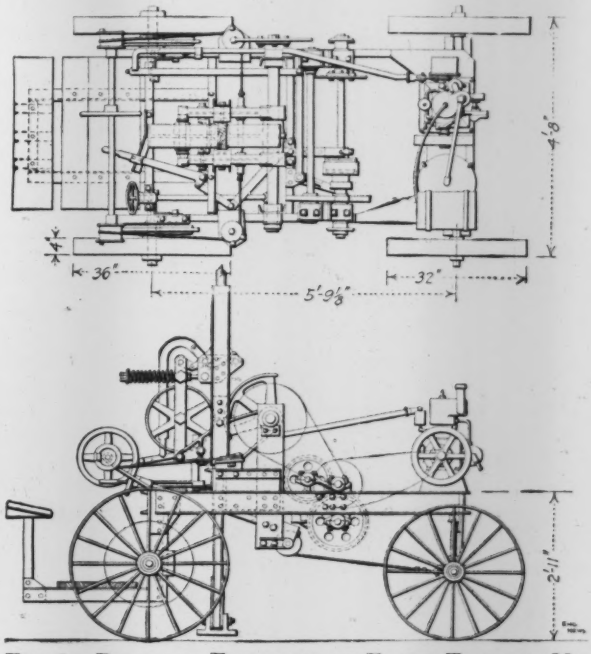


FIG. 2. PLAN AND ELEVATION OF EARTH TAMPING MACHINE

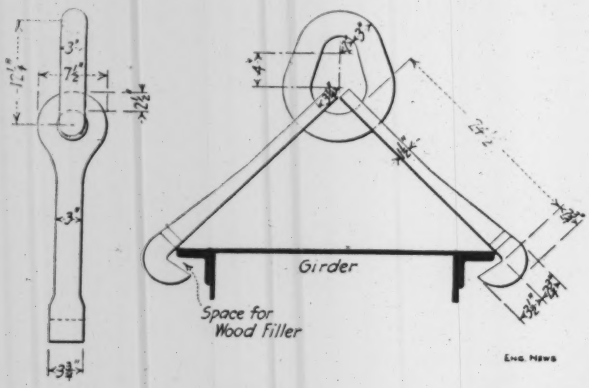
forward or steering axle is swiveled in a vertical plane to meet the same conditions.

This machine has been designed and built by Pawling & Harnischfeger, of Milwaukee, Wis.

Fire Hydrants for Benchmarks—The editorial and report in "Engineering News" of Apr. 30, 1914, relative to designs of water hydrants calls to mind an idea which has several times struck me very forcibly as being of some value. It is not an uncommon practice to use some part of a hydrant as a benchmark in leveling operations; the ordinary designs of hydrants contain several points (such as bolt heads), which are acceptable in their design for use as benches. Unless the point used is well described confusion and error are apt to result. While I do not approve of the use of hydrants as permanent benchmarks, I do believe that they often serve admirably as temporary benchmarks, hence my suggestion that if a lug or shoulder be cast on the hydrant, and so designed and placed as to serve as a rod point in wye-level work, some good would surely result therefrom. The lug or shoulder should have the letters BM cast thereon. I might go a step further and suggest that the hydrants themselves could be so constructed and set up as to serve even for permanent benches for all leveling save precise level work. —[H. C., M., Washington, D. C.]

Girder Hooks for Bridge Erection

For hoisting the girders of the Des Moines River Viaduct, the erection of which was done with a derrick car on the completed part of the deck, the hoisting hooks shown in the accompanying cut were used. They are of 20 tons capacity, and were found very satisfactory. Wood fillers were placed between the hooked ends of the bars and the flanges of the girder, as shown. Many of the girders weighed over 18 tons. The 50-ton derrick car carried a boom 80 ft. long.



GIRDER HOOKS FOR BRIDGE ERECTION; C. M. & ST. P. RY.

The viaduct is on a cutoff of the Chicago, Milwaukee & St. Paul Ry. between Madrid and Woodward, Iowa, and was completed in 1913. It is 2394 ft. long, with 40-ft. tower spans, 70-ft. intermediate spans and two 156½-ft. deck truss spans. It was designed and erected by the bridge department of the railway, under the direction of C. F. Loweth, Chief Engineer.

Special Cases of Fixed-End Beams

Nonuniform though distributed loading on beams involves complicated calculations, and handbooks give no formulas or other shortcut assistance. The complications are specially troublesome with beams fixed at the ends. An ingenious simplified method for fixed-end beams is offered by Bernhard Silberberg, Assistant Engineer, Board of Public Works, San Francisco.

METHOD—Taking the equations for end moments and reactions under a single concentrated load, and applying these to the load-element $p\,dx$ on a very short length dx of the beam, a single integration gives the desired result, since the end moment under the distributed load is merely the sum of the end moments caused by the separate load-elements acting separately.

EXAMPLE—Partial triangular loading, in which a length a at the left end is free from load, and to the right of this length the loading increases from zero to w at the right-hand end of the beam.

For a single concentrated load P distant x from the left end, the left-hand end moment and reaction are (from handbooks, or by calculation):

$$M_1 = \frac{P}{l^2}(2x^2l - x^3 - xl^2)$$

$$R_1 = \frac{P}{l^3}(l^3 + 2x^3 - 3x^2l)$$

These equations apply to every load-element $p\,dx$ for values of x from a to l; therefore, in the case in hand,

$$P = p\,dx = w\frac{x-a}{l-a}\,dx$$

$$dm_1 = \frac{p\,dx}{l^2}(2x^2l - x^3 - xl^2)$$

$$= \frac{w\,(x-a)}{l^2\,(l-a)}(2x^2l - x^3 - xl^2)$$

and

$$m_1 = \int_a^l \frac{w\,(x-a)}{l^2\,(l-a)}(2x^2l - x^3 - xl^2)$$

$$= \frac{w}{l^2\,(l-a)}\int_a^l [-x^4 + (2l+a)x^3 - (2al+l^2)x^2 + al^2x]$$

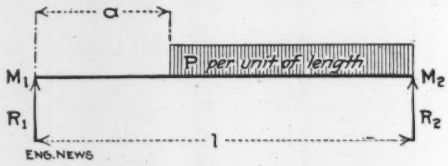
Integrating and reducing we get

$$m_1 = -\frac{w\,(l-a)^3\,(2l+3a)}{l^2}$$

The reaction may be found in the same way. The method is just as easily applied to distributed loading of uniform intensity but extending over part of the span only, as well as to distributed loading involving two or more intensities in different parts of the span.

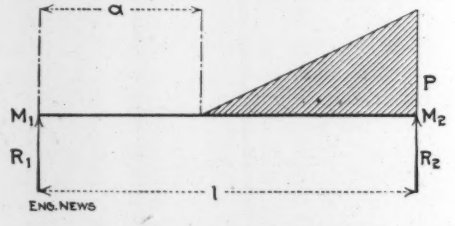
The following coefficients, computed by Mr. Silberberg, will be found sufficient to give values for most cases of partial loading on fixed-end beams. It will be understood that the cases tabulated may be combined in a variety of ways, forming more complex cases.

FIXED-END BEAMS UNDER PARTIAL UNIFORM LOADING



a	R_1	R_2	M_1	M_2
1/4 l	$\frac{135}{512}$ Pl	$\frac{249}{512}$ Pl	$-\frac{63}{1024}$ Pl²	$-\frac{81}{1024}$ Pl²
1/3 l	$\frac{16}{81}$ Pl	$\frac{38}{81}$ Pl	$-\frac{4}{81}$ Pl²	$-\frac{2}{27}$ Pl²
1/2 l	$\frac{3}{32}$ Pl	$\frac{13}{32}$ Pl	$\frac{5}{192}$ Pl²	$-\frac{11}{192}$ Pl²

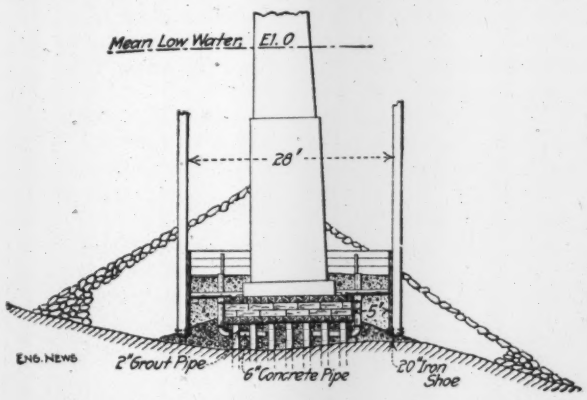
FIXED-END BEAMS UNDER PARTIAL TRIANGULAR LOADING



a	R_1	R_2	M_1	M_2
1/4 l	$\frac{189}{2560}$ Pl	$\frac{771}{2560}$ Pl	$-\frac{99}{5120}$ Pl²	$-\frac{201}{5120}$ Pl²
1/3 l	$\frac{22}{405}$ Pl	$\frac{113}{405}$ Pl	$-\frac{2}{135}$ Pl²	$-\frac{14}{405}$ Pl²
1/2 l	$\frac{1}{40}$ Pl	$\frac{9}{40}$ Pl	$-\frac{7}{960}$ Pl²	$-\frac{23}{960}$ Pl²

Repairing a Settling Bridge Pier

A rather extraordinary case of bridge-pier settlement, which occurred some years ago, has just been reported to the Boston Society of Civil Engineers in a paper read by James W. Rollins, a member of the contracting firm of Holbrook, Cabot & Rollins. The pier in question is Pier 4, the deepest of the pile-footed piers in the Brightman St. bridge at Fall River, Mass., across the Taunton Great River. This bridge is for highway only, has a 50-ft. roadway with two sidewalks of 10 ft. each, and three 200-ft. spans and a rolling-lift span with an opening of



REPAIRS TO PIER 4, BRIGHTMAN ST. BRIDGE, FALL RIVER, MASS.

100 ft. Pier 4 is between two 200-ft. spans, each of which weighs approximately 250 tons. It is a prismatic concrete structure 56 ft. high from the top of the pile footing to the top of the concrete masonry.

This pier, in common with two others in the bridge, was founded on timber piles which were driven into and sawed off at the rather hard river bottom, which is from 40 to 45 ft. below water level. The piers were built inside of timber cribs which were sunk to the pile footing by weighting with the masonry which formed the pier. After the pier was completed, the outside forms were removed, but the bottom of the crib, a thickness of 4 ft. in four layers of timber, was left to form a grillage floor between the piles and the masonry.

The bridge was completed in 1908. About three years later, decided settlement appeared in Pier 4. There was a settlement of about 2 ft. on one side of the pier and a list of 8 to 12 in. away from the side where the greatest settlement occurred. An investigation by divers showed that the broken stone filling which had been placed around the tops of the driven piles had washed out, and that the protruding piles had been broken off all along one side of the pier, allowing the pier to settle. On further investigation, it was found that the piles in this space between the river bottom and the crib had been eaten by the teredo, which is extremely prevalent in the waters of Fall River.

This particular activity of the teredo is, however, most extraordinary, if not unique. The U. S. Department of Agriculture Forest Service Bulletin, entitled "Preservation of Piling against Marine Wood Borers," says, in respect to the teredo:

> The portion of the pile commonly attacked is between mean low water and a point 4 ft. below, although sometimes it extends downward as far as the pressure of the water will permit. They seldom work to a depth of over 30 ft., and their worse attack is in very salty, warm clear water.

Mr. Rollins states that the bridge is in Taunton Great River, several miles from the sea and a city sewer discharges into the river within 500 ft. of the bridge, so that the water is not clear nor very salty, yet the teredo works there to a depth of 65 ft. below low water. The particular piles which caused the partial failure of the pier were eaten away at a depth of 45 ft. below low water.

REPAIRS TO THE PIER

The accompanying drawing shows the method taken to insure the future stability of the bridge.

It was necessary to proceed quite rapidly to the repair because of the condition of the entire bridge. The sinking of Pier No. 4 had thrown the spans upon that pier out of line and grade so far as to abut them against the next spans, with the result that a partial arch action was set up clear across the bridge. If the pier had sunk any farther it would probably not only have caused the failure of the spans next adjacent to it but possibly of the entire bridge. For this reason, the span most affected next to the pier was blocked up on a separate pile bent driven under the panel-point next to the pier.

Briefly, the method of repairing the pier was (1) to fill underneath it, that is, in between the protruding piles, with broken stone and gravel shoved in as hard as possible by divers; (2) to construct a framework around the pier with solid board forms about 8 ft. above the broken stone which had been placed under the pier, and to fill within this form with concrete; (3) to grout the stone under the pier and the new crib; and (4) to dump riprap around the pier proper. The methods in detail are described as follows by Mr. Rollins:

> First, 6-in. wrought-iron water pipes 6 ft. long were placed between the crippled piles under the grillage and elbows put on them, with vertical pipes about 8 ft. long fastened to them. A second set of grout pipes 2 in. in diameter were also put in, to make sure that the space between the grillage and the river bed should be made solid. The second operation was to build a form, of 3-in. planks about 8 ft. high, at the bottom of a long timber frame which extended to about 5 ft. above high water. This form was sunk 5 ft. outside of the old crib grillage floor, and had an iron plate on the lower edge to provide a seal into the bottom of the river or into the crushed stone which had been placed by divers subsequent to the discovery of the settlement of the pier.
>
> Riprap and crushed stone were then placed outside of this form to steady it and 1:2:3 concrete was deposited through a tremie from the top of the bridge into the space between the grillage and the form. This concrete was made wet to seal the bottom and the crushed stone and was brought up above the top of the grillage all around the pier. This was allowed to set up solid and then a rich concrete, made with 1-in. pebbles, sand and cement, was forced into the 6-in. pipe, but with no success, probably because the concrete deposited outside the grillage under considerable pressure had been forced into all the larger cavities under the grillage. The whole mass was then made solid by forcing grout through the 2-in. pipe until a pressure of 100 lb. per sq.in. would produce no flow.

Finally, a great amount of riprap was deposited all around the pier to a greater height on the weak than on the stronger side. The top of the pier was leveled up, the temporary supporting bent removed and the trusses fixed again on the new pier top.

The work was done by Holbrook, Cabot & Rollins, under a special contract. The total cost without any profit to the contractor, was about $60,000, repair work on Pier 4 alone costing about $25,000.

An Exceptionally Heavy and Complex Piledriver

A piledriver of unique design has been for some time in operation in the construction of a new reinforced-concrete pier for the Intercolonial Ry., of Canada, at the port of Halifax, Nova Scotia. The machine, two views of which are shown herewith, is exceptional in that it is of structural steel throughout and also in the flexibility of the movement of the leads on the barge to which the machine is attached.

The pier which is under construction is 800 ft. long and 235 ft. wide. It carries four railroad tracks running its full length. The entire construction is of reinforced concrete, designed for a safe live-load of 1000 lb. per sq.ft. on the third floor, and 500 lb. per sq.ft. on the second floor. The concrete superstructure is supported on 1800 reinforced-concrete piles 24 in. square, and ranging in length from 57 to 77 ft. These are heavily reinforced with eight steel rods varying in diameter, according to the length of the pile, from 1 in. to 1¼ in. In order to handle such extremely heavy and large piles effectively, special consideration had to be given to the type of piledriver, particularly as the bottom at the site was hardpan ranging from 3 to 14 ft. in thickness, through which the piles had to be driven to solid rock.

The contractor for the railway company, the Nova Scotia Construction Co., Ltd., of Sydney, N. S., called in W. L. Scott, Chief Engineer of Mussens, Ltd., of Montreal, to consult regarding the design of a machine which would meet the conditions. After investigating the matter carefully and laying out the general design of a floating piledriver, Mr. Scott took up the conditions with the Bucyrus Company of South Milwaukee, Wis., and this company contracted to design and build the piledriving machinery to be used in connection with the general layout and hull designed by Mr. Scott.

The design of the piledriver and of the entire mechanism, in fact, was controlled entirely by the conditions encountered. The minimum depth of water in the basin alongside the pier is 34 ft. at low tide, and at the outer end, 57 ft. At first, it appeared necessary to use piles ranging from 70 to 90 ft. long; the latter piles would have weighed approximately 30 tons apiece. Later it became apparent that so long a pile was not necessary, and a 77-ft. pile was substituted as a maximum length. These conditions fixed the height of the leads, which are about 76 ft. long over all, the top of the leads being about 70 ft. above the base. The driver, of steel construction, is fastened securely to a wooden hull 45 ft. wide, 108 ft. long, and 112 ft. deep. In order to support the lead mechanism, solid timber bulkheads are run fore and aft, and in addition there are several transverse trusses, all of which provide a hull of extremely rigid construction.

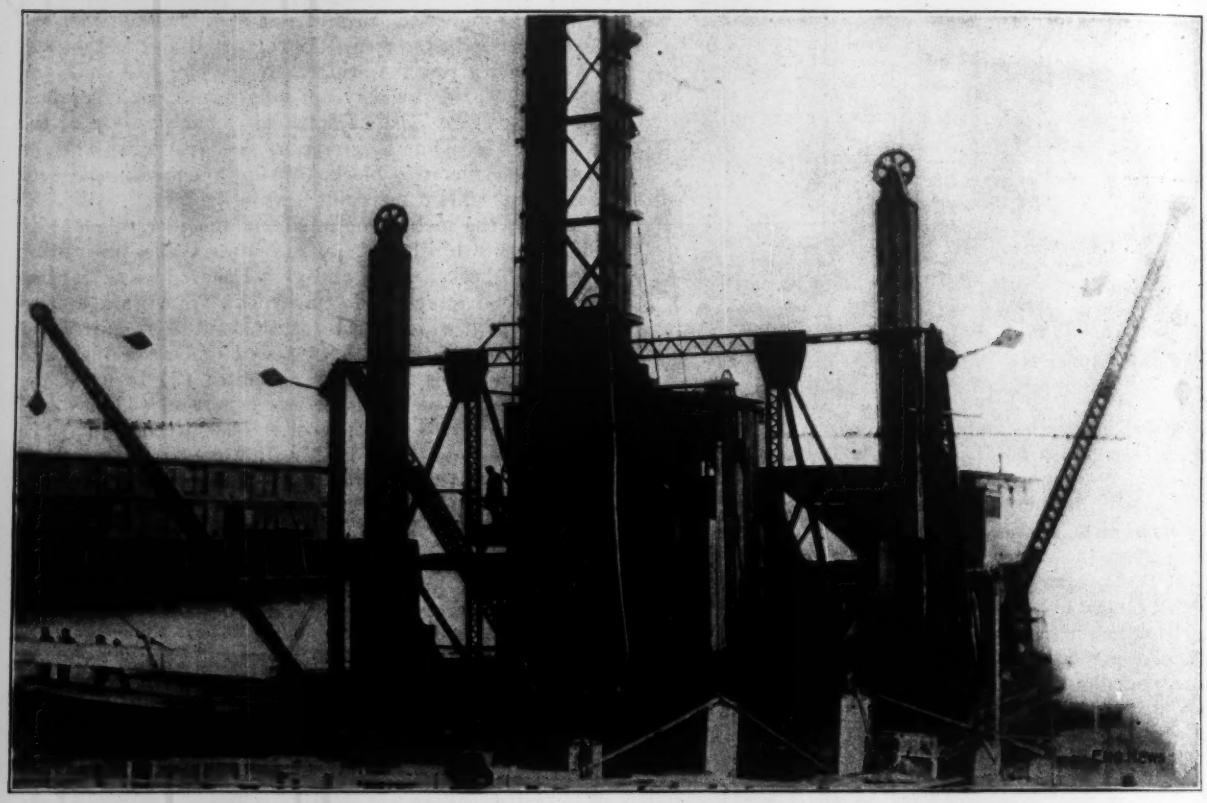


FIG. 1. STEEL PILEDRIVER WITH STEAM HAMMER FOR HALIFAX PORT WORK
[Manufactured by the Bucyrus Co.]

The piles for the pier were designed to be driven in clusters of straight piles with bracing inclined piles in each cluster. These inclined piles are on a batter of 15° and had to be driven close against the vertical piles. At several intersection points they are banded to these piles. It was extremely desirable that each one of the clusters,

with the inclined piles as well, could be driven without changing the position of the piledriver hull in the water. To meet this condition, the leads are designed so as to have a transverse motion of 8 ft. across the face of the hull, a fore-and-aft motion of 7 ft., and a transverse sloping position to an angle of at least 20°, sufficient in all to encompass the driving of a whole cluster of piles without moving the piledriver hull from the position to which it is fixed by the spuds. The motions of the driver are accomplished as follows:

The drums and machinery for hoisting the piles and the hammer and for operating the stern spud are driven by a double-cylinder 12x16-in. engine. All this machinery is mounted on a heavy structural-steel carriage, so arranged as to be moved fore and aft. This carriage is in turn mounted on rollers which travel fore and aft on a suitable track. The whole is moved by means of a rack-and-pinion drive from the main engine, the rack being rigidly bolted to the deck. The front of the carriage is provided with two transverse heavy structural-steel girders, one just above the deck level and the other some 30 ft. higher, running across the leads. These two transverse girders are plainly evident in Fig. 1. The upper girder carries a trunnion bearing which supports the weight of the leads. The lower carries a specially designed crosshead, attached to the leads in such a way that they are held vertically in any position.

The trunnion bearing on the upper girder and the crosshead on the lower one are each connected to independent screw shafts, governed by an 8x8-in. double-cylinder engine. These two screw shafts are also well shown in Fig. 1. When both screws are in gear with the engine, the leads can be moved transversely across the lead carriage. In order to swivel the leads it is only necessary to drive the upper or the lower screw alone. This screw design is to make the mechanism foolproof, as no false motion on the part of the operator can turn the leads over.

Two forward spuds hold the driver in position when in action. Each spud is provided with an independent spud-handling engine, which raises the spud and pins up the driver in the same fashion as a modern dipper dredge is operated. The spud machinery is placed well aft of the other machinery, and is connected with the spuds by wire rope leading over the top and bottom of each spud. The levers controlling the movements of the spud engines are so arranged that the engineer from his position in the traveling leader carriage has full control.

In addition to the lead carriage itself, each corner of the front end of the hull is provided with a steel derrick, which two derricks are interbraced, as shown in the views. These derricks are used in handling the piles. The piles are delivered to the driver loaded on scows, and are lifted toward the leads by the derricks, and a pile-hoist rope is then hooked into an eye-bolt set into the end of the pile, and it is swung into position from this rope which passes over a sheave at the top of the leads. An automatic dog locking device is provided at the top of the lead to carry the weight of the hammer while the pile is being placed in position, with also an intermediate stop to hold the hammer when the driver is not in use.

The steam hammer was designed and built by the Union Iron Works, of Hoboken, N. J., with an extremely large double-acting steam piledriving hammer, so far as we know, the largest ever constructed. The combined

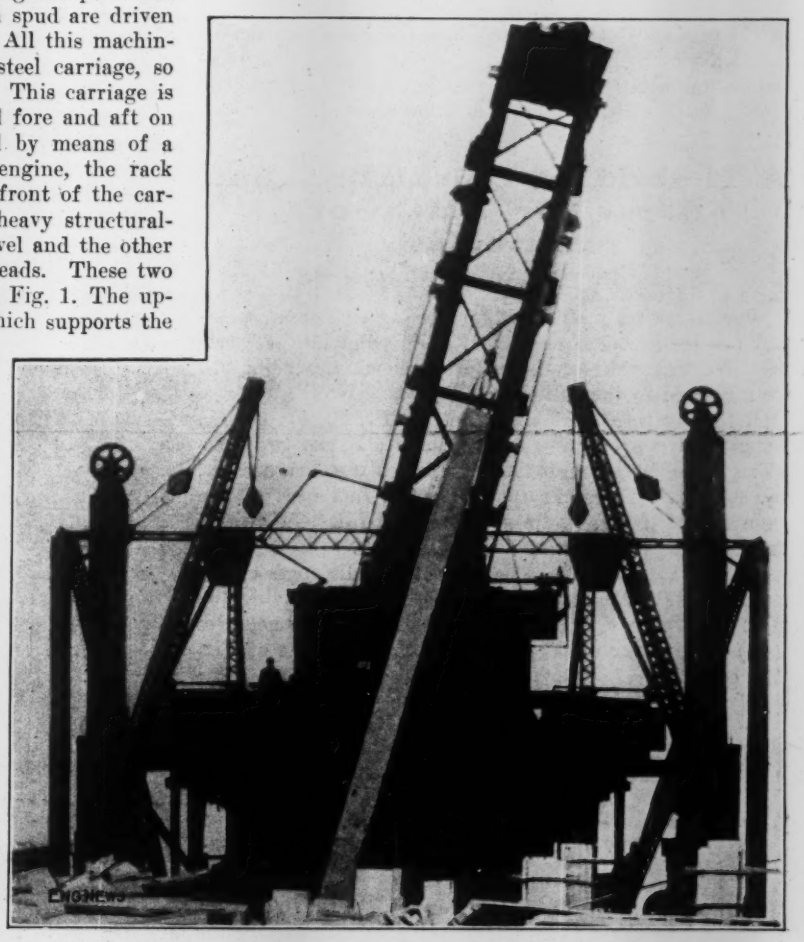


FIG. 2. VIEW OF BATTER PILE IN LEADS OF HALIFAX PILEDRIVER

weight of the hammer, with follower and follower guide is approximately 14 tons. The diameter of the cylinder is 14 in.; stroke, 36 in., and the weight of the ram alone is 4000 lb., with a mean effective steam pressure in the cylinder of 80 lb. per sq.in. The hammer is rated to develop 3,916,000 ft.-lb. per min., when it is striking 80 blows per min.

In order to take care of the reinforcing bars, which project 3 ft. beyond the end of the piles, the hammer is fitted with a special cap which passes over the reinforcement bars and bears directly on the concrete on the top of the piles. The leads are provided with special hammer trips, by means of which the hammer when hoisted can be switched in and out of the way of the swinging pile.

The machine was originally intended to drive one pile per hour, but it has been found that it can handle with ease 20 to 25 piles in a 10-hr. working day. The character of the material through which the pile is being driven controls the capacity considerably, as it takes from 25 to 30 blows of the hammer to drive the pile one inch into the hardpan which is encountered. In actual practice, however, it has only been necessary to drive about 15 piles a day.

The pier at Halifax, as stated above, is being built for the Intercolonial Ry. John Kennedy, of Montreal, Can., is Consulting Engineer. The Nova Scotia Construction Co., Ltd., of Sydney, N. S., is the contractor. The pile-driver was built by the Bucyrus Co., with the advice of W. L. Scott, Chief Engineer of Mussens, Ltd.

A Method of Calculating End Areas for Earthwork Formulas*

By E. S. M. Lovelace†

The following method of calculating end areas, without the reduction of elevations or the plotting of cross-sections, using direct the notes taken in the field, has been devised by the author.

Referring to Figs. 1 and 2, let P_0, P_1, P_2, P_3, P_4, P_5, be any polygonal figures and OX and OY any two lines at right angles to one another; and let the distances a_1, a_2, a_3 . . . . represent the horizontal distances of the points P_1, P_2, P_3 . . . from the vertical lines OY, which in the examples given include P_0 and P_5.

Let h_0, h_1, h_2, h_3 . . . . be the vertical distances of the points P_0, P_1, P_2, P_3 . . . from the lines OX giving the plus (+) sign to the distances below, and the negative (—) sign to the distances above OX. The area of Fig. 1 is equal to:

$$\tfrac{1}{2}\,[a_1\,(h_2 - h_0) + a_2\,(h_3 - h_1) + a_3\,(h_4 - h_2) + a_4\,(h_5 - h_3)]$$

And the area of Fig. 2 is equal to:

$$\tfrac{1}{2}\,[a_1\,(h_2 - h_0) + a_2\,(-h_3 - h_1) + a_3\,(-h_4 - h_2) + a_4\,(-h_5 + h_3)]$$

or

$$\tfrac{1}{2}\,[a_1\,(h_2 - h_0) - a_2\,(h_3 + h_1) - a_3\,(h_4 + h_2) - a_4\,(h_5 - h_3)]$$

In words, the areas of the polyons P_0 P_1, P_2, P_3, P_4, P_5, are found by taking one-half of the algebraic sum of the products found by multiplying consecutively each horizontal distance a_1, a_2, a_3 . . . . by the algebraic difference between the ordinates of the points coming immediately after and before the point corresponding to the given horizontal distance.

Proof of the foregoing is easily derived by drawing the horizontal dotted lines AP_1, BP_2, CP_3, DP_4 and the diagonal dotted lines BP_1, CP_2, DP_3.

Then for Fig. 1, we have the area equal to:

$$\tfrac{1}{2}\,a_1\,(h_2 - h_0) = \tfrac{1}{2}\,a_1\,BP_0 = \tfrac{1}{2}\,a_1\,(BA + AP_0) = \tfrac{1}{2}\,a_1\,BA + \tfrac{1}{2}\,a_1\,AP_0 = \textit{area of } BP_0P_1$$

$$+\ \tfrac{1}{2}\,a_2\,(h_3 - h_1) = \tfrac{1}{2}\,a_2\,CA = \tfrac{1}{2}\,a_2\,(CB + BA) = \tfrac{1}{2}\,a_2\,CB + \tfrac{1}{2}\,a_2\,BA = \textit{area of } CBP_1P_2$$

$$+\ \tfrac{1}{2}\,a_3\,(h_4 - h_2) = \tfrac{1}{2}\,a_3\,DB = \tfrac{1}{2}\,a_3\,(DC + CB) = \tfrac{1}{2}\,a_3\,DC + \tfrac{1}{2}\,a_3\,CB = \textit{area of } DCP_2P_3$$

$$+\ \tfrac{1}{2}\,a_4\,(h_5 - h_3) = \tfrac{1}{2}\,a_4P_5C = \tfrac{1}{2}\,a_4\,(\quad + DC) = \tfrac{1}{2}\,a_4\,P_5D + \tfrac{1}{2}\,a_4\,DC = \textit{area of } P_5[\quad]P_4$$

the sum of which equals the area of $P_0\,P_1\,P_2\,P_3\,P_4\,P_5$.

For Fig. 2, the proof is similar, for using the minus (—) signs for the ordinates (corresponding to a fill or embankment), the same analysis may be made.

To adapt the method to fieldwork, so as to obtain the areas of the cross-sections from the level notes direct; let the horizontal line OX be the height of instrument and the line OY a vertical line through the center line. The ordinates h_0 h_1 h_2 h_3 . . . are rod readings taken at various points on the section, and the horizontal distances a_1 a_2 a_3 . . . . are the corresponding distances right or left of the center line. The areas to the right and left of the center line are calculated separately and added; or, of course, the horizontal distances a_1 a_2 a_3 . . . on the left could be given a minus sign, and the results included in the general formula.

Thus in Fig. 3, by the formula, the area right

$$= \tfrac{1}{2}\,[6.0\,(2.4 - 2.9) + 13.0\,(1.9 - 1.6) + 20.4\,(8.8 - 2.4) + 10.0\,(8.8 - 1.9)]$$

$$= \tfrac{1}{2}\,[-6.0 \cdot 0.5 + 13.0 \cdot 0.3 + 20.4 \cdot 6.4 \cdot 10.0 \cdot 6.9] = 100.23$$

area left

$$= \tfrac{1}{2}\,[8.0\,(7.1 - 2.9) + 12.5\,(8.8 - 5.0) + 10.0\,(8.8 - 7.1)]$$

$$= \tfrac{1}{2}\,[8.0 \cdot 4.2 + 12.5 \cdot 3.8 + 10.0 \cdot 1.7] = 49.05$$

Total end area = 149.28

In working out these areas in the fieldbook, the rod readings or ordinates can usually be subtracted mentally,

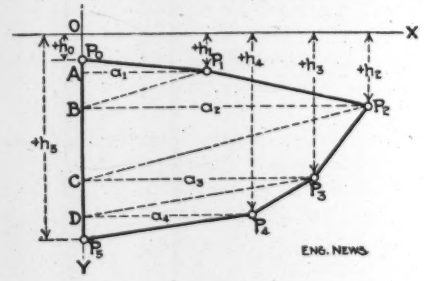


Fig. 1.

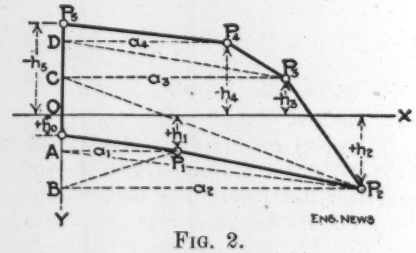


Fig. 2.

and their differences at once jotted down with the proper sign.

In case it is impossible to complete the section with a single setting of the instrument, that is, a change in the H. I., it is only necessary after setting up to find the difference between the new H. I. and the old H. I., and the rod readings corrected and referred to the old H. I., as the OX line. For example, in Fig. 4: Find on the upper side of the section the difference 3.2 between 103.54, the new H. I., and 100.34, the original H. I.

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TABLE II. SOME FIELD NOTES AND END AREA CALCULATIONS

Station	C. or F.	B.S.	H.I.	F.S.	Elevation	Grade	H.I. to grade	Calculations, remarks	End areas Excavation	End areas Embankment
			100.34					Road bed 20 ft. exc. 16 ft. bank		
R	+6.9		1.6 2.4	(1.9)				—6.0, 0.5+13.0, 0.3+20.4, 6.4+10.0, 6.9		
			6.0 13.0	(20.4)				2		
41	+5.9	2.9				91.50	+8.8		149.3	
		0						+8.0, 4.2+12.5, 3.8+10.0, 1.7		
L	+1.7		5.0 (7.1)					2		
			8.0 (12.5)					—8.0, 0.8+15.5, 3.5+10.0, 3.7		
R	+3.7		5.5 (5.3)					2		
			8.0 (15.5)							
41+40	+2.9	6.1				91.30	+9.0		65.2	
		0						+6.0, 2.2+11.0, 2.3+10.0, 0.7		
L	+0.7		6.7 (8.3)					2		
			6.0 (11.0)						0.0	0.0
41+62	0.0		Grade			91.19		—7.0, 1.2—10.4, 2.4—8.0, 1.6		
R	—1.6		11.7 (10.9)					2		
			7.0 (10.4)							
42	—2.8	12.1				91.00	+9.3			73.2
		0						+9.0, 2.6—16.1, 5.0—8.0, 5.4		
L	—5.4		14.3 (14.7)					2		
			9.0 (16.1)							
T.P.		0.84	89.12	12.06	88.28			On rock 40 ft. right of 42		
R	—3.0		2.4 (1.4)					—9.0, 1.6—12.5, 4.0—8.0, 3.0		
			9.0 (12.5)					2		
42+60	—4.6	3.0				90.70	—1.6			127.8
		0						+5.0, 1.9+10.0, 1.8—17.7, 6.5—8.0, 6.5		
L	—6.5		3.1 4.9	(4.9				2		
			5.0 10.0	(17.7)						

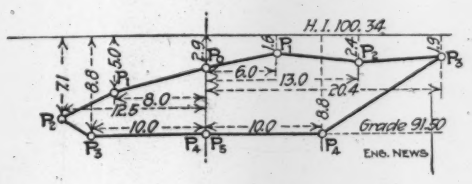


FIG. 3.

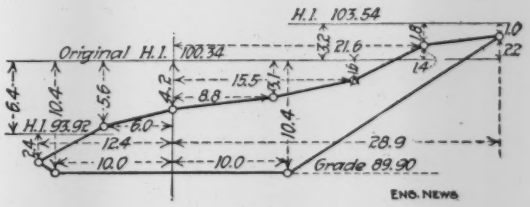


FIG. 4.

The rod readings reduced are —1.4 and —2.2, referred to the original H. I. On the lower of left-hand side of the section the new H. I, 93.92, being lower than 100.34, the original H. I., their difference 6.4 must be added to

TABLE I. SOME OF REDUCED NOTES

Station	C. or F.	B.S.	H.I.	F.S.	Elevation	Grade
			100.34			
R	+12.6		3.1 1.6	See page 32	—1.4	(—2.2)
			8.8 15.5		21.6	(28.9)
40	+6.2	4.2				89.90
		0				
L	+1.6		5.6	See page 32		(8.8)
			6.0			(12.4)

2.4, the rod reading as taken, giving a reading of 8.8, referred to the original H. I.

If, however, there are a number of consecutive cross-sections requiring changes of H. I., it is better to enter on another page of the level book, under their respective H. I., the rod readings as taken, and then on the marginal page, reduce them as shown above with reference to the original H. I. The original cross-sections can then be completed by bringing back the corrected readings. Table I gives the notes for the cross-section, Fig. 4, reduced.

The resulting end area would then be: area right

$$= \tfrac{1}{2}\,[8.8\ (1.6 - 4.2) + 15.5\ (-1.4 - 3.1) + 21.6\ (-2.2 - 1.6) + 28.9\ (10.4 + 1.4) + 10.0\ (10.4 + 2.2)]$$

$$= \tfrac{1}{2}\,[-8.8 \cdot 2.6 - 15.5 \cdot 4.5 - 21.6 \cdot 3.8 + 28.9 \cdot 11.8 + 10.0 \cdot 12.6] = 146.15$$

area left

$$= \tfrac{1}{2}\,[6.0\ (8.8 - 4.2) + 12.4\ (10.4 - 5.6) + 10.0\ (10.4 - 8.8)]$$

$$= \tfrac{1}{2}\,[6.0 \cdot 4.6 + 12.4 \cdot 4.8 + 10.0 \cdot 1.6] = 51.56$$

Total end area 197.71

These and further explanations are published by the author in an eight-page booklet designed to be bound in the back of an ordinary fieldbook.

An Australian Method of Setting Slope Stakes

BY SAM M. SUTHERLAND*

In looking over some back numbers of ENGINEERING NEWS, I found a description of an improved method of setting out batter pegs for the top edge of a cutting in sloping ground. It is rather a troublesome operation to place a peg at the exact intersection of the batter slope and of the natural surface, so a method used here that I have never seen described anywhere may be of interest to your readers.

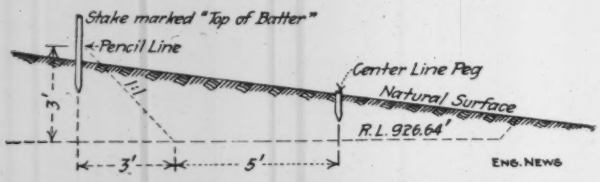
Instead of marking the line where the batter slope cuts the natural surface, a stake about 18 in. or 2 ft. in length is driven a short distance back from the top of batter, and at a whole number of feet measured horizontally from the foot of batter. The face of the stake toward the cutting is roughly dressed with a tomahawk, and a horizontal pencil line is drawn where the plane of the batter cuts the stake. Under the line, a vertical arrow is drawn to distinguish it, and the stake is marked "top of batter."

The center line chainage is usually written on the stake. In scooping out the excavation roughly, the actual edge can be marked well enough by the workman with a foot rule; for trimming the corner of the template is placed on the pencil line.

For example, an excavation has to be set out with a bottom width of 10 ft. at R. L. (Reduced Level of sub-

*Dandenong, Victoria, Australia.

grade) 926.64 ft. and with side slopes of 1 on 1. The level reading is 3.14 ft. on a benchmark whose reduced level is 931.25 ft., giving R. L. of collimation line = 934.39; and the height of the collimation line above bottom of the excavation = 934.39 — 926.64 = 7.75 ft. Then at 5 ft. left of center peg the reading of staff held on batter would be 7.75 ft.; at 6 ft. left of center peg reading of staff, 6.75 ft.; at 7 ft. left of center peg reading of staff, 5.75 ft.; at 8 ft. left of center peg reading of staff, 4.75 ft.



AUSTRALIAN METHOD OF SETTING SLOPE STAKES

The staff is first held at 7 ft. left of the center peg and the reading is 5.40; the staff is then moved a foot further from the center peg as a second trial and the reading is about 5.27; a stake is driven here at 8 ft. left of center peg, then the staff is held against the face of stake and moved slowly up until the reading is 4.75. A pencil line is ruled across the stake at the bottom of the staff, thus marking the intersection of the plane of the batter slope with the stake.

Batters can be set out very quickly by this method, and it has the advantage that the stakes are clear of the excavation and need not be disturbed; and the finished excavation can be easily checked. The stakes are also very convenient for picking up rough chainages after the center line pegs are destroyed.

NOTES

Experimental Pavement of Small Wood Blocks—Last fall a strip of 100 ft. of experimental paving was put down on one of the residential streets of Tacoma, Wash., using creosoted wood blocks only, 1⅞ in. wide, 2 in. deep and 3⅞ in. long. All blocks were surfaced on one side and one edge to these dimensions, and are of Douglas fir. The blocks were treated in retorts with approximately 18 lb. per cu.ft. of creosote oil. It is in the treatment that the chief advantage is claimed for the small blocks, for excellent penetration without excessive treatment and a uniformity of grain are obtained more easily than with blocks of standard size.

The blocks were laid on a concrete base, 14½ ft. wide, and were laid without an expansion joint of any kind. The pavement was divided into four sections each 25 ft. long, as follows:

(1) Blocks laid with long axis at right angles to the line of traffic on a 1-in. sand cushion; broken joints, blocks rammed tight.

(2) Blocks laid at an angle of 67½° to line of traffic on a 1-in. sand cushion, rammed tight and joints broken as in Section 1.

(3) Blocks laid on a 1-in. cushion made of one part of cement to three parts of fine sand mixed and spread dry; blocks have long axis parallel to the curb, making unbroken joints along the line of traffic; at right angles to the curb joints are broken.

(4) Blocks laid in the same manner as in Section 3, except bedded in a one part cement to three part sand grout mixed wet enough to take a smooth trowel finish.

An inspection of the pavement some months after it was laid showed water oozing out between the blocks of Section 4, when the blocks were stepped on. The blocks were not perfectly bedded on the grout for there was a perceptible movement of the surface. The other sections were in excellent condition.

The blocks were furnished and laid by the creosoting department of the St. Paul & Tacoma Lumber Co., on their application for permission to do so as a part of an asphalt-paving contract which was under construction. The lumber company gave the city of Tacoma an agreement to replace the blocks with asphalt when so ordered. The work was under the direction of R. H. Wegener from the creosoting department of this company. [From F. H. Cushman, Assistant Engineer, Department of Public Works, Tacoma, Wash.]

Strength of Band and Spiral Hooping—A reinforced-concrete pile at White Plains, N. Y., which was driven to rock but on the engineer's request was further driven, crushed near the top in the manner shown by the view herewith. The head of the pile had band reinforcement, 1x⅛-in. bands 2 in. on centers; just below the reinforcement was No. 3 spiraling, laid "close," i.e., about ½-in. to 1-in. on centers. The spiraled hooping broke, as the view shows, while the band hooping stood up. The Cummings Structural Concrete Co., Pittsburgh, Penn., who supplied the view, give the belief that this occurrence demonstrates band hooping to be superior to spiral hooping. Other deductions are possible, however, and there is opportunity for interesting speculation on this case of crushing.

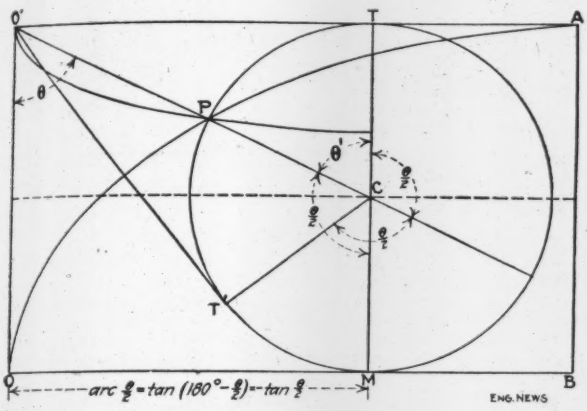
SPIRAL HOOPING OF REINFORCED-CONCRETE PILE. BURST BY OVERDRIVING

Removing a Reinforced-Concrete Bridge Pier with Dynamite—In the removal by blasting of a reinforced-concrete bridge pier 37 ft. long, 30 ft. wide at one end and 17 ft. wide at the other and 16 ft. thick, which had fallen over on its side and was blocking a river channel in Indiana, the dynamite was placed in a rather unusual manner. In half-bushel market baskets mud was laid on the bottom and sides; dynamite was then placed on the bottom mud and covered with more mud. The baskets were then lowered against the concrete pier and the dynamite discharged. This method was successful in pulverizing the concrete so that piles could be driven through it, and new piers constructed on the site of the old ones.

A Solution of the Kite-Shaped Race-Track Problem—Readers of "Engineering News" of 25 years ago will recall that race-track surveying problems were then quite live issues. In spite of laws adverse to the sporting fraternity, horse racing continues to some extent; and surveyors are still occasionally called upon to lay out race tracks. For the benefit of these and for surveyors who are interested in an academic way, we offer a solution of a race-track problem by Robert C. Thomas, professor of mathematics and engineering at The Citadel, Charleston, S. C.

The problem of laying out a kite-shaped race course, composed of an arc of a circle and two intersecting tangents to the ends of the arc, may be solved graphically by the inter-

section of a cycloid and a conchoid. The conditions require that one-half the arc shall be equal to the tangent of the supplement of half the angle of the arc. The half arc must be greater than a quadrant to be equal to a tangent; therefore, to find the radius and angle of the arc we must find the intersection of the cycloid and the conchoid (the point P in the accompanying figure), whose pole is at O′, and whose equation is $\rho = r\ (\sec \theta' - 1)$, the polar axis being O′O and $\theta' = (180° - \frac{\theta}{2})$.



Draw O′P to intersection with a line parallel to the base of the cycloid and at a distance r above it. This gives C, the center of the arc, making the arc PM = tan OM = O′T = O′T′.

The angle whose arc is equal in length to the tangent of its supplement is 116° 14′, to the nearest minute, and OM = O′T = 2.029 r = r tan 63° 46′ = r arc 116° 14′. Hence $r = \frac{O'T}{2.029}$, or approximately one-half the tangent, so the radius can be determined for any desired length of tangent or arc.

The angle may also be found by plotting the locus of $y = \theta$, and the locus of $y \tan (180° - \theta) = -\tan \theta$; the intersection of the two loci gives $\theta = 116° 14'$.

Hand-Operated Percussion Drill—A new type of small hand-operated percussion drill is shown in the accompanying illustration. As the shaft and pinion D and the spider E revolve, two tool-steel hammers F swing out by centrifugal force, and at each revolution strike the end of the drill rod, the force of the blow throwing the hammer back so that it passes clear of the rod and then swings out again to deliver another blow on the next revolution. The gear ratio is 1: 6¼, so that with the two hammers there are about 13 blows on the drill for each revolution of the handle. The operator can run it at 75 (or more) revolutions per min., and as the force of the blows varies with the speed, he can regulate the work of the drill very closely.

The machine is made in two sizes, one taking ⅛-in. to ⅝-in. drills, and the other taking ½-in. to 1¼-in. drills. An extension rod can be fitted to the handle to brace the machine against a ceiling or between beams, where it cannot be held up to its work conveniently by the operator. The machine has been named the "Drilhammer," and is made by the Orr & Lockett Hardware Co., of Chicago.

A similar drill, in which the blow was delivered by springs, was described in our issue of Nov. 21, 1912, p. 961.

Driving 1⅛-in Field Rivets, Equitable Building—The field connections in the lower seven stories of the Equitable Building, New York City, are very largely made with 1⅛-in. rivets, of grip up to about 9 in. It has been found that to drive these rivets successfully the largest obtainable pneumatic hammers are required, and air pressures up to 110 lb. must be used. Further, it is important that the point of the rivet be no hotter than the shank under the head, and in some cases the rivets are cooled at the point by dipping into water.

In the early stages of the work, a rather heavy percentage of rivets was condemned by the inspector, in some cases because not enough stock had been left for the heading up. As these rivets are now being used, a length of 2⅛ in. is allowed over the grip to form the head and to supply metal for the upset. This allowance is somewhat excessive, but it gives a safe margin to allow for any overrun in the size of hole, etc.

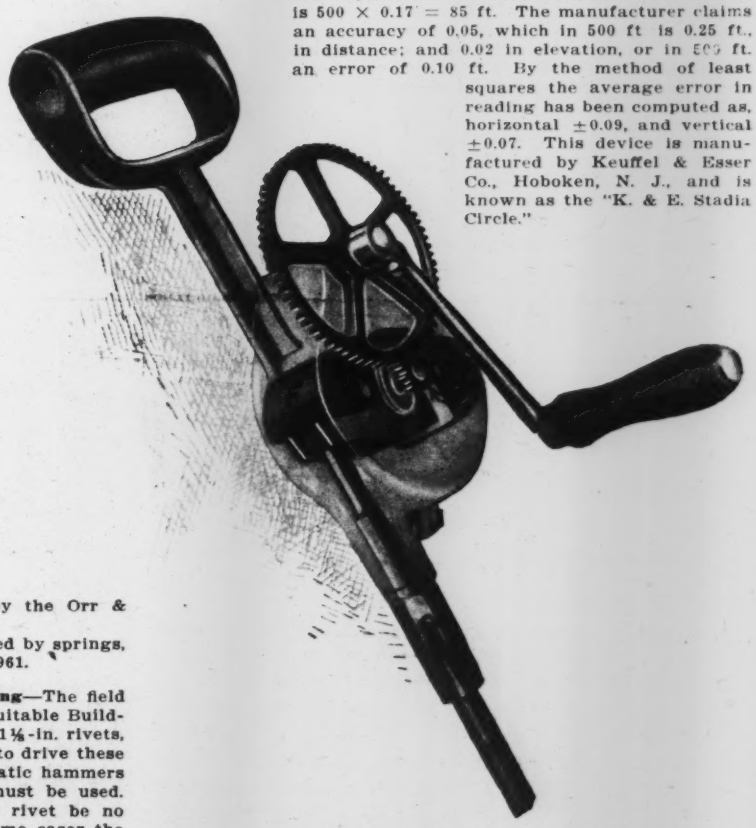
Practically all the field-connection holes are reamed out after assembling, in spite of the fact that they are already drilled holes. The reaming insures a perfect match and guards against difficulty in entering the hot rivet.

The hammers used are long-stroke Boyer and Ingersoll-Rand $1^{13}/_{64}$x8-in. hammers.

The experience of the riveting gangs of the Thompson-Starrett Co. as stated by the boss of riveters, is that about 50% more ⅞-in. rivets, or about 75% more ¾-in. rivets could be driven in the time required for a given number of 1⅛-in. rivets. This, as will be seen, means that the 1⅛-in. rivets do not cost more per sq.in. to drive than rivets of smaller size.

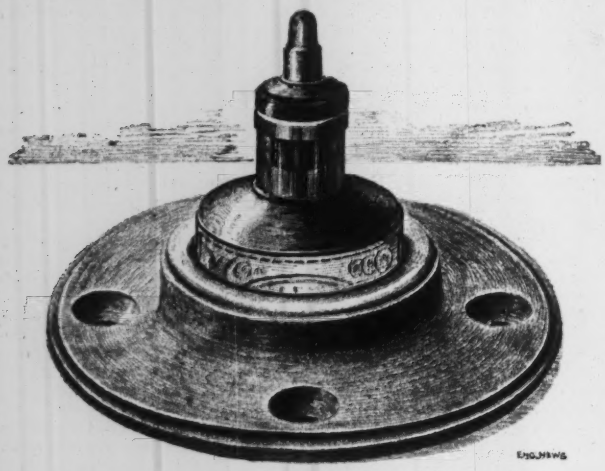
New Stadia Circle for Transits and Alidades—A new device to shorten stadia work has recently been put on the market, consisting of a modification of a regular transit or alidade circle, in which the degree graduations on two diametrically opposite segments are replaced by special graduations, which give directly the percentage to be applied to the observed stadia distance to obtain the true distance and the difference in elevation. An arc for measuring vertical angles, with a vernier between the legs of the left-hand standard, is the same as an ordinary transit arc. For about 60° on either side of the horizontal diameter of the vertical circle are the special stadia percentage graduations, with indexes on the right and left. On the left-hand index marked HOR. is read the percentage factor to be applied to the observed stadia distance by rod intercept to obtain the correct horizontal distance. On the right-hand index marked VERT. is read the percentage factor to be applied to obtain the difference in elevation.

For instance, suppose the observed stadia distance to be 500 ft. and the HOR. index reading 0.97 and the VERT. index reading 0.17. The true horizontal distance is 500 × 0.97 = 485 ft.; and the vertical difference in elevation is 500 × 0.17 = 85 ft. The manufacturer claims an accuracy of 0.05, which in 500 ft is 0.25 ft., in distance; and 0.02 in elevation, or in 500 ft. an error of 0.10 ft. By the method of least squares the average error in reading has been computed as, horizontal ±0.09, and vertical ±0.07. This device is manufactured by Keuffel & Esser Co., Hoboken, N. J., and is known as the "K. & E. Stadia Circle."



HAND-OPERATED PERCUSSION DRILL

An Improvised Ink-Bottle Holder—An ingenious New York draftsman has discovered that a 1½-in. steam-pipe floor-plate makes an ideal holder for a standard-size drawing bottle. The thread in the floor-plate is tapered slightly, so that the bottle does not go clear through, but is wedged in such a way that the bottle sticks so tightly that the plate can be lifted by taking hold of the bottle. The floor-plate illustrated is a standard pipe fitting made by the Nason Manufacturing Co., 71 Fulton St., New York City.



AN IMPROVISED INK-BOTTLE HOLDER

Dismantling the Kingsbridge Armory Erection Travelers—The removal of the two large two-boom timber derrick-towers used for erecting the steelwork of the Kingsbridge Armory ("Eng. News," Jan. 22) constitutes quite a task. All the heavy steelwork has been erected so that the big travelers are no longer required. Each is 91 ft. high, 60x60 ft. in plan and equipped with two separately operated derricks. The framing is all 12x12-in. timbers, except the front 14x14-in. corner posts, which carry the derricks. One traveler was taken down during the week of Apr. 20; the demolition of the other commenced on May 1.

In dismantling a traveler, one of the derricks on the top lowered the other to the ground. A jinnywinch was then installed in its place and lowered the second derrick. A gin-pole was set on the lower chords of the last two arch trusses and proceeded to take down the framing. When low enough, the traveler was pulled under the trusses to which blocks and falls were fastened. The work then went forward rapidly.

A Socketed Connection for Steel I-Beams—A rivetless connection for steel I-beams, devised by Wesley Wait, Newburgh, N. Y., was the subject of a test in the Columbia University testing laboratory recently. Fig. 1 shows the principle of construction of the joint. The horizontal member or cap has its flanges slotted to fit a tapered notch cut in the top of the post in line of the web (removing a length of the web); and the joint is made by slipping the cap into the notch in the post. The bent tested is shown by the view Fig. 2. It was rather crudely made, and the notches were not tapered, or only slightly tapered, as the picture shows. Loads up to

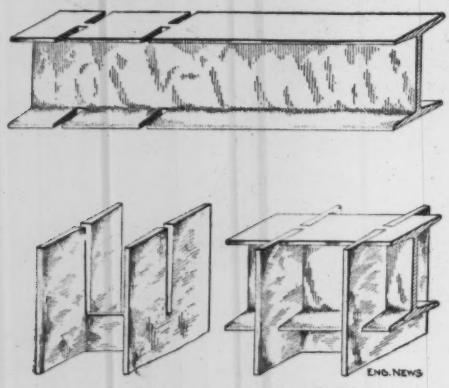


FIG. 1. WAIT JOINT FOR I-BEAMS

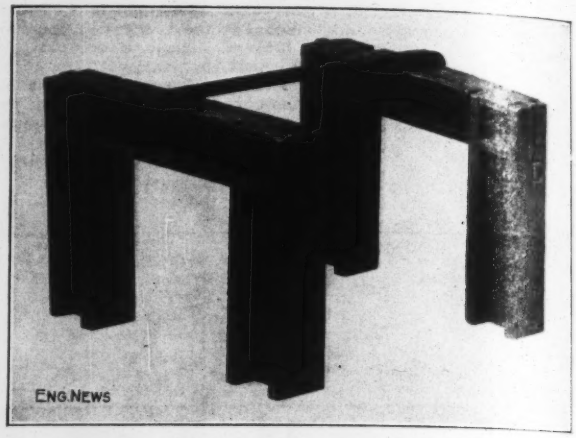


FIG. 2. TEST BENTS WITH THE WAIT JOINTS

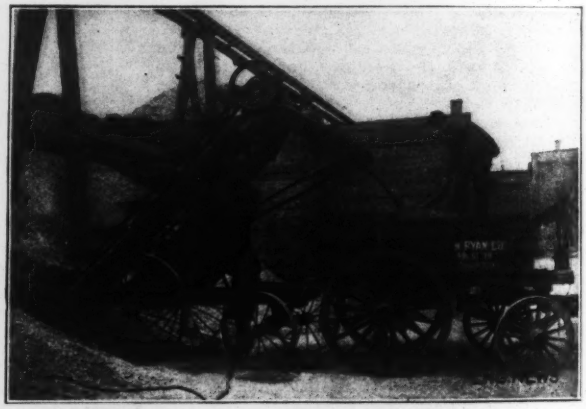
45,000 lb. were applied (by loading blocks at the quarter-points of the cap), and no weakness of the connection was evident, although the yield-point of the cap-beams was passed.

The joint is thought to have possibilities for use in steel mine sets, particularly on account of the quickness and ease of connecting up.

Steel Bridges Erected Without Shop Paint—The Pittsburgh & Lake Erie Ry. in some new bridge construction at Youngstown, O., specified that no shop paint or oil be put on. The first paint is applied after erection, scraping the steel clean before painting. The object is to allow the mill scale to loosen by rusting, and thus get the paint applied on the steel itself.

An Electrically Operated Portable Wagon Loader of the bucket conveyor type is illustrated herewith. No small gasoline engine has yet been found which is dependable enough to give constant service with the kind of labor ordinarily available for operating the loader, so the use of the loader shown is confined to yards or to city work where electric current, either direct or alternating, is to be had. The motor is 7½ hp. inclosed in a dust-proof casing.

When not in use and when being moved from place to place the loader is tipped down so as to rest on the bottom of the trapezoidal frame containing the motor. When tipped up as shown the conveyor buckets are moved into the pile of material by sliding on the channel frame of the carriage, by means of a hand lever working a pawl and ratchet. There is 3 ft. of this movement.



AN ELECTRICALLY-OPERATED PORTABLE WAGON LOADER

The overall height of the loader in collapsed position is 7 ft. 6 in.; when operating its overall height is 13 ft. The width is 5 ft. 11 in. and the length 12 ft. The two rear wheels are 60 in. in diameter and the front wheels are 42 in. The machine illustrated is loading crushed stone from a storage yard pile. It can also be used to advantage in sand and gravel pits and for coal handling. The loader shown is manufactured by the George Haiss Manufacturing Co., Inc., 142nd St. and Rider Ave., New York City.

Editorials

The Extension of Motor Omnibus Service

From all indications the present season is to witness a very large development in the use of motor omnibuses for transportation in country districts and also in city suburbs. Applications for franchises to operate omnibus lines are made to the Public Utility Commissions of many states at almost every meeting. In New York City, the applications made a year ago for motor omnibus franchises in the Borough of Manhattan have never been acted upon by the Board of Estimate and are still pending. Recently the Hillside Transportation Co. has applied for franchises to operate four routes in the Long Island suburbs of New York City, having a total extent of about 18 miles. The Chief of the Bureau of Franchises, Harry C. Nichols, has recommended that the franchises be granted on condition that they may be revoked at any time. He urges that the city cannot fail to learn something of the effect of the operation of motor busses on road surfaces and of the capabilities, efficiency and value in transportation service of different types of the motor bus. The company has already ordered six vehicles, which will be delivered next month. The possible development of motor-omnibus service was discussed at length in an editorial in our issue of Nov. 20, 1913. Such motor omnibuses are already running in some of the New Jersey suburbs of New York City and there is every reason to expect, with reasonable coöperation on the part of municipal officials, that their use will rapidly extend.

⁂

A Homeric Nod

The editors of ENGINEERING NEWS have never laid claim to infallibility; they are willing to admit complete ignorance of a number of things and proneness to error in a few more. But as concerns knowledge of the Panama Canal they have hitherto regarded themselves rather highly. For fifteen years they have followed with sleuth-like persistence the progress of that great work from its inception. Almost weekly they have recorded in these columns some item regarding the construction now fast approaching completion on the Isthmus. If a high mark on an examination on the Panama Canal had been a requirement for the job of Governor of the Zone, they might have run Col. Goethals a close race for the appointment.

This consciousness of complete knowledge is rudely shattered by the mistake in last week's issue which credited the new Hamburg-American liner "Vaterland" with being the only vessel unable to pass the Panama locks because of its 100-ft. beam. As the editors now realize only too keenly, and as many attentive readers have promptly informed them, the Panama locks have a clear width of 110 ft., and the "Vaterland" could pass freely from Colon to Panama, if by any chance such an unexpected voyage were required of her.

A concerted lapse of memory is responsible for the error, which is not an excuse but an explanation, and while the editors are sorry to have spread abroad misinformation, it has doubtless done them good to realize anew the truth of the ancient maxim that it is human to err.

⁂

The Dangerous Encroachments on the North River Channel

Regularly every few years the United States Army Engineers in charge of the harbors and rivers around New York City recommend to their lay superiors in the Department of War at Washington that the encroachment of steamship piers upon the navigation channel of the North River be finally and definitely restricted. From time to time there has appeared at the head of the Department a man who could recognize the authority with which these engineers spoke, and who upheld them in their locally unpopular campaign. But invariably the pressure brought to bear upon the Department by the business and political influence of New York City has resulted in a so called "temporary" permission to extend the piers a hundred or more feet out into the steadily narrowing channel.

Up to the present time, these "temporary" permits have not differed in any way from a permanent permit; that is, they still exist and there seems to be no effort being made to end their existence, and the direct and flagrant opposition to all sane engineering advice, in which the steamship and political authorities in New York City persist, has been eminently successful.

The fairway in the North River opposite the piers of the Hamburg-American Line in Hoboken was given in 1911 as something over 3500 ft. Since that time, piers on either side of the river have been extended 100 ft. each so that the fairway is now about 3300 ft. An investigation prior to 1911 showed that the steamships of that day maneuvering to enter the slips between the piers required a minimum of 1300 ft. Since that time, the transatlantic ships have increased in length nearly 200 ft., so it is fair to assume that the required fairway for the ships, which probably increased at a greater ratio than the increase in length, is at least 1600 ft. Obviously, under these conditions, two of the large ships on opposite sides of the river could not maneuver at the same time without serious danger to themselves.

In the opposition to the increase in length of piers, however, the port engineers did not so much consider the possibility of danger to the maneuvering large vessels, because it can be arranged that two such ships need not enter the piers at the same time; but they did consider the possibility of danger to the many other craft which ply the waters of the North River. Every foot which is decreased from the fairway from pierhead to pierhead adds a percentage of danger to the river traffic.

The truth of this conclusion could not have been more definitely exemplified than it was a week ago, when the

huge liner "Vaterland" was warped into its berth at Hoboken for the first time. As noted in our news columns, for four hours a large fleet of powerful tugs pulled and pushed the 58,000-ton bulk in an almost vain effort to turn its bow into the slip beside the recently extended pier at Hoboken. Only by the narrowest margin were several serious accidents avoided. Tugs with tows were hardly dragged out of the way of the tide and current-drawn liner, and at one time a heavily laden ferry boat just avoided being crushed between the steamship and the adjacent pier.

The difficulties of maneuvering so large a vessel were, if anything, more clearly shown by the behavior of the "Vaterland" in leaving New York on May 26, as also reported in our news columns. The 3300-ft. fairway was only just sufficient to permit her turning, and that on a slack tide.

The local authorities, influenced by a ridiculous fear that the steamship companies may abandon New York for some other port, have up to the present time indirectly, at least, opposed a permanent restriction of the fairway; but the experience of this maiden voyage of the "Vaterland" should convince them that expenditures to move the pierhead line shoreward are imperative. It is the height of absurdity that the normal traffic of the harbor of New York City, the traffic which brings in possibly 90% of the commerce to the city, should be held up for hours and subjected to very serious danger for the occasional record-breaker whose value to the city is largely in the advertising it gives.

The Westchester Revelations

A year ago, in our issues of June 5 and June 12, 1913, we described the Westchester Ry., a high-speed electric suburban line, 26 miles long, completed in the early part of 1913 at the northern part of New York City. Last week this railway came into national prominence through the revelations of ex-President Chas. S. Mellen, of the New York, New Haven & Hartford R.R., concerning the taking over of this enterprise by the New Haven Company. The facts, as stated by Mr. Mellen in his testimony before the Interstate Commerce Commission are briefly as follows: There had long been two rival railway projects in the territory adjacent to the New Haven railway line in northern New York, one known as the Westchester, and another known as the Port Chester. Mr. Mellen had always regarded these schemes as more or less in the nature of strikes, that is to say, of schemes which were projected by those promoting them with the idea that they would be eventually bought off by the New Haven Company. Railway financial history is full of just such enterprises.

Mr. Mellen never regarded these railway projects as anything in which the New Haven Co. ought to take any serious interest, until Oakleigh Thorne, a prominent New York financier, called upon Mr. Mellen and told him that he, Thorne, had gained control of both these competing schemes, and asked Mellen to take up the matter of their purchase with the directors of the New Haven Co.

Shortly afterward the late J. P. Morgan, a director of the New Haven Company, moved in a directors' meeting that a committee of directors be appointed to consider the purchase of these projects from Mr. Thorne. Morgan, Wm. Rockefeller and one other director were appointed as such a committee. The utmost value represented by both projects at that time was roughly $4,000,000. Very little construction had been done, but some valuable real estate and rights-of-way had been acquired. After a year or more Mr. Morgan reported for this committee that they had expended something over $11,000,000 of the New Haven's money in acquiring the Westchester and Port Chester enterprises. As to how the money was spent, or who received it, Mr. Mellen professed total ignorance.

This was by no means the worst, however, for following the purchase of the franchise, rights-of-way, etc., at this enormous figure, the New Haven Company proceeded to construct the road. A length of 26 miles has been completed and put in operation, and is, without doubt, one of the most expensive, not to say extravagant, pieces of railway construction, in the country, if not in the world. According to Mr. Mellen's statement, it represents a total investment by the New Haven of about $35,000,000. As illustrating the folly of the enterprise, Mr. Mellen stated that last year (a year when the New Haven R.R. Co. had precious little money to spare) nearly $1,250,000 had to be taken from the New Haven treasury to pay the deficit by which this $35,000,000 railway which it controls failed to earn its operating expenses and fixed charges.

It is little wonder that, under these circumstances, Mr Mellen referred to the exchange of New Haven stock for stock of the Westchester project which was made in 1907, by saying that "we exchanged our valuable stock worth $150 a share, for stock of the Westchester worth about ten cents a pound!"

The above are the facts as made of record in the Interstate Commerce investigations; but the question which is left unanswered is what were the governing motives that led Messrs. Morgan and Rockefeller to expend the New Haven Company's funds in this way. Was there private graft, to benefit this or that or the other interest, concealed in this operation? If that were the sole explanation, then it is difficult to see why so many millions were poured into the extravagant construction of the Westchester road.

It is, however, quite compatible with the other financial exploits in connection with the New Haven Corporation, and for that matter also with other financiering with which the Morgan firm had been identified, to conceive that these hard headed financiers really believed that the properties which they purchased at these extravagant figures would eventually prove worth what had been paid for them. There was a great deal of such financiering done in the carnival of trust formation during the years from 1900 to 1907, and the experience of buying properties at two or three times their cost value, and then selling them to the public at seven or eight times their cost value, had unsettled the judgment of a good many men who posed as "Captains of Industry" and financial magnates a half-dozen years ago.

A point of great interest to engineers in this exposure is that it cannot fail to have the greatest influence in destroying public confidence in the banker as a safe business guide. No financier has ever had to such a high degree the confidence of the investing public as did Mr. Morgan; but the New Haven revelations coupled with the disastrous experiences of his firm as promoters of the shipping and shipbuilding combinations and other enter-

prises which have grounded on the shoals illustrate that the banker whose overconfidence or conceit causes him to rely on his own judgment rather than the opinions of competent experts is truly a blind man leading the blind.

There were dozens of engineers who could have shown by a careful analysis of the Westchester enterprise when it was taken over by the New Haven that there was no prospect of profit if it were carried through to completion at all commensurate with the risk and the large investment involved. In fact, there were such men in the service of the New Haven R.R., from President Mellen down.

No other class of men in the community has given as careful study to the problems involved in the operation of railways and other public utilities as have engineers. It is true that there are all kinds of engineers and one often finds the names of engineers as sponsors of enterprises of doubtful merit. But it is the bankers' or business men's responsibility to pick the engineer as an adviser whose opinion can be trusted just as he selects his legal adviser. A conservative banking concern should no more assume responsibility for an enterprise which they offer to the investing public without the favorable opinion of an engineer than they would sell bonds whose validity was questioned without obtaining the advice of an attorney.

The Cost of the Panama Canal as Estimated and as Built; a Triumph for the Engineering Profession

There is no task laid upon the engineer which involves greater responsibilities and greater difficulties than the making of preliminary estimates of cost. On the accuracy and reliability of such estimates the decision of the most important engineering questions often turns. The estimate of the cost of the Panama Canal work involved special difficulties. The work was of a magnitude wholly unprecedented. It had to be carried on at a distance of two thousand miles from the base of supplies. The influence of tropical conditions, of Governmental control, of uncertain labor markets, all had to be taken into consideration with a view to their possible influence upon the cost. The work had to continue over a term of years, during which large changes in market prices of labor and of materials were probable.

Bearing all these things in mind, it is of greatest interest to compare the engineers' estimates of cost of the Panama Canal per unit of work with the actual final cost, and to find that as regards the largest items the cost is very close to what was originally estimated.

We say "per unit of work," which is, of course, the only way in which a fair comparison may be made. The total cost of the canal is, it is true, far more than was contemplated by the engineers who made the estimates for the work; but the chief reason for the excess of cost is that a much larger canal has been built than was originally planned when the cost estimates were made; and it is well to note also that none of the engineers who made the original plans and estimates for the Panama Canal have had anything to do with the actual construction. The canal recommended by the Commission of 1899 was to have a depth of 35 ft. and a width of 150 ft. The locks were to be 740 ft. in length and 84 ft. wide. The minority members of the International Commission of 1905, whose plan was adopted for the canal, increased the depth to 40 ft. and the width to 300 ft., and the size of the lock chambers to 95 ft. in width and 900 ft. in length. Through the deepest part of Culebra Cut where the cutting was heaviest, the width was made 200 ft.

When the construction work was well under way, the width through the Culebra cut was increased by order of President Roosevelt to 300 ft. and the lock dimensions were increased 110 ft. width and 1000 ft. length.

The Commission of 1899 estimated the total cost of the Panama Canal at $144,000,000 in round numbers.* The minority members of the International Commission of 1905 estimated the cost at a little under $140,000,000. The "Canal Record" of Apr. 29 contains a detailed summary of the construction expenditures on the canal from the beginning of the work to Dec. 31, 1913, and the total is $198,000,000. This includes administrative and general expenses and the cost of the plant which has been used in the construction, which cost has been now nearly all absorbed by depreciation charges. This figure does not include, however (nor did the engineers' estimates above quoted), the expenditures for sanitation, hospitals or civil government of the Canal Zone, which items have amounted to a much larger sum than anyone expected when the canal work was undertaken.

A comparison between the estimates of cost per unit of work made by the eminent engineers who controlled the Commissions of 1899 and 1905 and the actual expenditures upon the canal is so interesting and makes such a remarkable showing for the engineering profession that it deserves wide publicity.

The biggest single item in the cost of the canal has been the digging of the Culebra cut, involving in round figures about 110,000,000 cu.yd. of excavation. The Commission of 1899 estimated the cost of removal of the soft rock in Culebra cut at 80c. per cu.yd. The International Commission of 1905 appointed a special committee to fix unit prices, made up of Messrs. William Barclay Parsons, Isham Randolph and J. W. Welcker of Holland. This committee's estimate was as follows: "Culebra cut, all material without classification above Elev. +10, 80c. per cu.yd." The committee therefore fixed upon the same figure as the commission of 1899 for the cost of the Culebra work.

Turning now to the results of experience, the official statement above referred to shows that up to Dec. 31, 1913, the total cost of all dry excavation in the section from Gatun to Pedro Miguel, which includes the Culebra cut, was slightly under $86,000,000, and the average cost was 78.18c. per cu.yd. In other words, the engineering estimates made ten and fifteen years ago of the cost per cubic yard of excavating Culebra cut, the largest item in the entire canal work, varied by only some 2% from the actual cost of the work.

The next largest item in the cost of the Panama Canal is the concrete in the locks. The estimate for concrete in place made by both the Engineering Commissions of 1899 and 1905 was $8 per cubic yard. The period since

*The estimate made in 1899 of $144,000,000 for the construction of the canal was based on a total excavation of about 100,000,000 cu.yd. The various increases in the size and capacity of the canal which have been made since then have increased the total excavation in connection with the canal and its auxiliary works to over 200,000,000 cu.yd.

these estimates were made has witnessed a large reduction in the cost of concrete, partially due to a reduction in the cost of cement and partly to improvements in machinery for preparing the aggregate and for mixing and placing the concrete. The Gatun locks contain over 2,000,000 cu.yd. of concrete, costing over $15,000,000, and their cost per cubic yard was $7.4641. In the locks at Pedro Miguel a considerably better cost record was made. The cost per cubic yard for something over 900,000 yd. of concrete was $5.8897 per cubic yard. In the Miraflores locks, which contain about 1,500,000 cu.yd. of concrete, a still better record was made and the unit cost was reduced to $5.3665.

It is fair to say, however, that whereas the original estimates above quoted included all the concrete in the locks at $8 per cubic yard, the engineers on the isthmus have classified separately the concrete which was laid in connection with the machinery of the locks. This concrete has cost about $12 per cubic yard and as the total expenditure on such concrete in all the locks is three-quarters of a million dollars, it would tend to increase somewhat the unit price of the other concrete if all the concrete were classed together.

Engineers will agree that the original estimates have come astonishingly close to the cost of the finished concrete in the locks when all the many changes which have taken place in conditions since the estimate was made are taken into consideration.

Another very large item in the cost of canal work is dredging, especially in the harbors at the Pacific and Atlantic entrances. The Commission of 1899 estimated the unit cost of excavating dredgable material at 20c. per cubic yard. The Committee on Unit Prices of 1905 reduced this for the mud and sand in Colon and Panama harbors to 15c. per cu.yd. but estimated 25c. per cu.yd. for "soft material removed by hydraulic pumps in canal."

Turning now to the records of actual cost we find that all the dredging done from Gatun Locks to the sea, including that through the Mindi swamp as well as in the shallow water of Limon Bay, has amounted to about 38,500,000 cu.yd. and the unit cost has been 23.25c. On the Pacific end about 40,000,000 cu.yd. of material have been excavated by dredging between Pedro Miguel and the sea, most of which has been excavated in the sea level section from Miraflores locks to Balboa. The average cost of all this work has been 25.52c.

It must be remembered in comparing these estimates with the figures of actual cost, that the figures of actual cost include the dredging of rock under water after it has been broken up by drilling and blasting or other means. This sort of excavation was separately classified in the engineers' estimates so that the actual cost of dredging all the material in the sea-level sections at each end comes exceedingly close to the preliminary estimates made by the engineers.

Another very large item in the cost of the Panama Canal work is the great dam at Gatun, in which dam over 12,000,000 cu.yd. of dry filling were dumped from cars and nearly 11,000,000 yd. were placed by hydraulic filling. The commission of 1899 estimated 60c. per cu.yd. as the cost for embankments and backfilling. The construction of dams by hydraulic fill was not then contemplated. The commission of 1905 estimated for dams as follows: "Earth borrowed, 50c. per cu.yd.; extra price for placing in dams from line of canal within a distance of four miles, 20c. per cu.yd. When placed in dam by pumps from barges below Elev. +50, 20c. per cu.yd.; above elevation +50, 30c. per cu.yd." In actual construction the dry filling has cost less and the hydraulic fill a little more than these estimates. The average cost per cubic yard for the 12,000,000 yd. of dry filling has been 43.55c., and for the hydraulic filling, 29.8 c.

Another large item in the cost of the work was the excavation at the lock sites. The commission of 1899 estimated the cost of removal of hard rock per cubic yard $1.15. The commission of 1905 used the same figure of $1.15 for hard rock excavated in the dry elsewhere than in the Culebra Cut, and in the lock sites below Elev. —10, the estimate was $1.50 per cu.yd.

In actual experience on the Gatun locks over 4,600,000 yd. were excavated in the dry, nearly all of which was in rock that would be classified as hard rock and the average cost was only 67.71c. Dredging excavation was also done in the Gatun locks amounting to nearly 1,800,000 cu.yd. at an average cost of 32c. per cu.yd. There are also items for preparing foundations by excavation and dredging amounting to nearly 250,000 cu.yd., the average cost of which was $2 per cubic yard. This latter item doubtless includes some of the material for which a higher price was allowed, as noted above, by the commission of 1905.

On the locks at Pedro Miguel the dry excavation cost 91.2c. per cu.yd. The item "preparing foundations," involving 176,000 cu.yd. of excavation, cost $2.72 per cu.yd. At the Miraflores locks, dry excavation in the lock site amounted to 93c. per cu.yd., dredging excavation 46c., while 416,000 yd. were excavated under the head "preparing foundations" at a cost of $1.90. The general comparison, therefore, shows that the engineers in charge of the work on the isthmus succeeded in excavating the rock at the lock sites for a considerably lower figure than was allowed by the engineers who made the original estimate.

There is another estimate which was made by the minority members of the International Commission of 1905 and which it is of great interest to compare at this time with the final result. That is the estimate of the time required for completion of the lock canal. This was one of the matters which influenced the decision as to the choice of the sea-level or the lock plan.

The minority members, Messrs. Alfred Noble, Isham Randolph, F. P. Stearns, Gen. H. L. Abbot and H. L. Ripley estimated at the beginning of 1906 that the lock canal which they recommended could be completed in 8½ years. Their opinion was that the controlling factors in the time of completion would be the Culebra Cut and the Gatun locks, and their opinion was that the excavation of the cut could be completed in a somewhat shorter time than the lock construction but that the whole work could be completed and opened for traffic in 8½ years.

This report of the Board was dated on Jan. 10, 1906, and the period of 8½ years from that time would mark the middle of 1914 as the time for the completion of the canal. As our readers are aware, the work on the canal is now so far completed that vessels are passing through it in advance of its formal opening. Had it not been for the enormous slides which occurred on the sides of the Culebra Cut, the prediction of the minority board of 1905 would have been exactly fulfilled; and the Culebra Cut would have been completed considerably in advance

of the locks. These slides, however, so prolonged the work on the cut that the locks were completed first.

We believe these figures, showing the remarkable accuracy of the engineers' estimates made in advance of the undertaking of the greatest engineering work in the world and under conditions where the unknown factors made the work of accurate estimating peculiarly difficult, ought to be more widely known. The public which employs engineers has a great distrust of engineers' estimates and does not realize how closely competent engineers, thoroughly familiar with the work to be undertaken, can predict in advance the entire cost which a given work planned on standard and well established lines will involve. That the actual cost of the Panama Canal, per cubic yard of earth and rock excavated and per cubic yard of masonry built, is very close to the estimate which was made by the engineers before the work was undertaken is a fact which ought to be spread before the public by the engineering profession through every possible channel.

Letters to the Editor

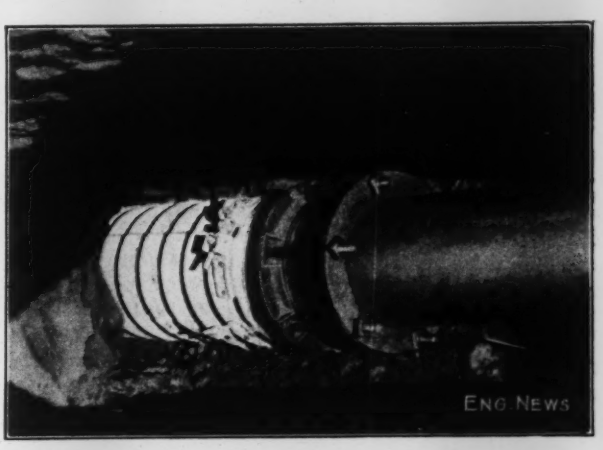
Corrugated-Iron Culverts for Protecting Pipe Crossings

Sir—I noted the very interesting article in your issue of April 9, on the storm damage to pipes of the Midway Gas Co., in southern California. I expected to see some mention made of the unique scheme of protecting the pipe lines from external load where crossings under railways and highways had to be made. As I have not seen this commented on, I would like to add a supplemental note to the article mentioned.

As shown in the accompanying figure, the gas main was slipped into a corrugated-iron culvert, 12 in. in diameter, and both buried together. It proved entirely successful and was an appreciable factor in reducing the time of construction of this line—which point was of great importance.

B. G. MARSHALL.

National Corrugated Culvert Manufacturing Co.,
Middletown, Ohio, May 13, 1914.



CORRUGATED CULVERT PROTECTING A GAS-MAIN CROSSING

A "Roast" for an Engineer

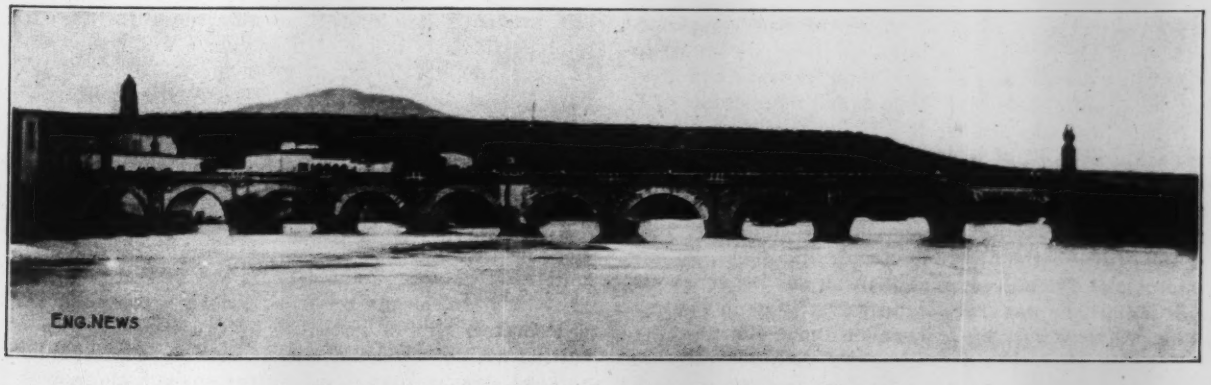
Sir—Referring to your editorial "An Estimate Wide of the Mark," in ENGINEERING NEWS, of Jan. 29, 1914, p. 251, I inclose a photograph of the fine old masonry bridge over the Lerma River at La Piedad, Mexico. At one end of the bridge there are some quaint inscriptions in Spanish, including this:

For this work in 1807 the principal neighbors offered to contribute 40 thousand pesos. The charter having been obtained from the Viceroy, the work was not built, because the estimate was 60 thousand pesos. And in 1834 the work was completed for less than one quarter of the amount of the estimate.

The "neighbors" evidently blamed the fellow that made that estimate for the long delay in building the bridge. To have one's errors carved on stone in a prominent place ought to make an engineer careful of his estimates.

T. K. MATHEWSON.

Guanajuato, Mexico, Apartado 50, Apr. 26, 1914.



BRIDGE OVER THE LERMA RIVER AT LA PIEDAD, MEXICO

Another Government Railroad

Sir—In your issue of May 7, 1914, under the title, "The Alaskan Railway Engineers," it is stated that one of these engineers has been "engineering and administrative head of the *only* government railway."

The U. S. Reclamation Service is operating, and has for more than two years been operating, a standard-gage government railroad 19 miles in length in the State of Idaho. This railroad—the Boise & Arrowrock R.R.—was built by the Government in 1911 and extends from Barberton on the Oregon Short Line to the site of the Arrowrock Dam, now under construction, on the Boise River.

Trains are operated each way daily and the road has handled about a million ton-miles of freight traffic and over 45,000 passengers.

A. P. DAVIS,
Chief Engineer, U. S. Reclamation Service.
Washington, D. C., May 18, 1914.

A Proposed Monument to the Late Alfred Noble

Sir—I desire to add to Hugh L. Cooper's tribute to Alfred Noble (ENGINEERING NEWS, Apr. 23) that those who did know Mr. Noble well personally felt not only the most intense admiration and respect for his professional abilities but also a very deep and sincere affection for the man himself. I worked for Mr. Noble several times, first in 1884-85 and last in 1899-1901; and from close personal contact with him then and since I am sure that there is just cause to regard him even higher as a man than as an engineer.

He was especially kind and helpful to young engineers, and to work under him was equivalent to a good engineering education.

He was modest and unassuming to an unusual degree and never made the least effort to advance his own interests. At the same time, he had a strong fund of quiet humor; and he had carefully considered opinions on all public questions.

He was especially well read on historical and international questions relating to great engineering works, such as the Panama Canal; and only last November he deplored the then apparent probability that the United States would break a treaty obligation by the not repealing the law to exempt our coastwise ships from payment of Panama Canal tolls.

He was very just and unbiased in all his dealings. On one occasion, even after a contractor had complained bitterly of Mr. Noble's rulings, on later becoming involved in a dispute and an important arbitration proceeding, he at once chose Mr. Noble as his representative.

In his professional life, Mr. Noble was very unusual in having kept all of his proficiency in the lower classes of engineering work unimpaired throughout all his advance to world renown in the consideration of the most important engineering problems of the time. In 1888, he was still a better instrumentman, draftsman, letterer, computer and detail designer than any of his assistants; and in 1901, he was still able to use calculus in his work as an ordinary working tool, and without effort.

His field of work was much wider than generally was realized, even by engineers. He was recognized chiefly as probably the foremost engineer of the United States and of the world in matters dealing with deep and difficult subaqueous foundations, especially those using the penumatic process; but he was also equally able in the field of steel superstructure and in nearly all classes of hydraulic work relating to civil engineering.

In all this wide range his grasp of each kind of work included not only the broadest general considerations, such as are necessary in determining the selection of the most advisable from among several alternative projects, but also all the elements and details from the first preliminary conception to the final completion. He was even a good organizer and executive on construction work and carried out a number of difficult pieces of foundation work, masonry and superstructure erection, without the assistance of contractors.

His judgment was clear, reliable and safe as to the probable cost of work; and he not infrequently worked up bids for such work with contractors, and sometimes he had general supervision of the carrying out of their work.

Mr. Noble's highest quality, however, was a sane, safe, sound judgment on all matters connected with civil-engineering problems. I believe the attainment of such a judgment to be the highest and most important achievement possible to an engineer; and that such a trained judgment is of more use and value to the world than any mere knowledge can be, no matter how thorough. I believe Mr. Noble to have possessed such a sound engineering judgment to an almost unprecedented degree.

In recent years, Mr. Noble received many public honors, such as the Presidency of the American Society of Civil Engineers, the Presidency of the American Institute of Consulting Engineers, the John Fritz Medal, and honorary degrees from universities.

I greatly hope that the engineers of the United States will at the same time honor not only Mr. Noble but also the profession of civil engineering by the erection, as suggested by Mr. Cooper, of some suitable memorial to perpetuate to future generations the memory of the life and professional work of Alfred Noble.

EDWIN DURYEA, JR.
San Francisco, Calif., May 15, 1914.

Rodmen on the Panama Canal

Sir: In several recent issues I have noted a discussion going on relative to the salaries of a "rodman" on the Panama Canal work. I fully agree with "A. P. C." that "C. E." and "H. M. C." have read into the words "college graduate" meanings that were never intended. I refer particularly to the civil engineers who come to the Isthmus and are forced to work for $83.33 until they have proved themselves. "H. M. C." speaks of "inexperienced college graduates of engineering schools," but if he will remember correctly, the Commission stipulated that men with no practical experience need not apply.

I know nothing about clerical work, being strictly a "field man," but, like "A. P. C.," I would like to see any clerk on the "Zone," with his minimum $125 salary, run a party of "blacks" for ten hours out in that jungle. I know what it means to drink swamp water, and play peacemaker, with a level-rod, to two negroes, bent on cutting each others heart out.

I had occasion once to get dinner at the little mess-hall

out on the Gamboa dumps and fell into conversation with a locomotive engineer. He expressed his thoughts pretty freely and on parting he told me, "If I ever so much as caught my son looking at one of those machines (here he pointed to my "gun"), I'd kill him." That expresses pretty well the feeling of the other employees on the "ditch" toward a rodman's job.

I hope that when the time comes to offer positions on this new Alaskan Railway, that the man who has to do the "dirty-work" will get a more even split with the office force.

I'm not a kicker and I enjoyed my work done on the "Zone," but I don't believe that "C. E." and "H. M. C." could have been "rodmen," or their letters would have been different I'm sure.

W. B. W.

New Haven, Conn., May 20, 1914.

The Bactericidal, Deodorizing and Physiologic Action of Ozone

Sir—The comments upon our paper, "The Bactericidal, Deodorizing and Physiologic Action of Ozone,"* which have appeared in letters to the ENGINEERING NEWS and other journals indicate that some of the writers have not carefully read our report or else that they have failed to see the object of certain experimental procedures. We are not concerned with irrelevant criticism and intentional misrepresentation, but sincere misunderstanding is entitled to an explanation.

It should be clear to all who read our report carefully that our experimental procedures fall into two groups: (1) Those upon the bactericidal, deodorizing and physiologic action of ozone in sufficiently great concentration to give such action—a phase of the subject primarily of scientific interest; (2) those upon the possible bactericidal, deodorizing and physiologic action of ozone in such concentrations as are permissible in ventilation—a phase of the subject essentially of practical interest.

The injurious action of strong concentrations of ozone on animal life was established long ago. Our physiological experiments with these stronger concentrations were designed *to elucidate the mechanism of this injurious action on man and experimental animals, and not to prove the fact of the injurious action itself or to combat the practical use of ozone in ventilation.*

We were careful to point out the difficulties in the way of demonstrating either any beneficial or any injurious action of ozone on man in such concentrations as can obtain in practical ventilation, difficulties similar to those encountered in determining the action of small quantities of food preservatives, alcohol, or other drugs on man. The test must extend over a long time (years) and this increases the disturbing factors over which the experimenter has no control. Our test with weak ozone on 22 animals extended over a period of four weeks. So far as we know these observations cover a longer period than any other adequately controlled experiment with ozone. Nevertheless, we carefully refrained from basing any final conclusion on these tests.

No *adequately controlled* tests showing any beneficial action of ozone in disease have so far been put on record. The published accounts consist in the main of each writer's impressions without objective criteria, and without considering the factors of spontaneous recoveries and improvements due to other therapeutic measures. (No one treats any malady with ozone alone.)

As a destroyer of germs in the human body, ozone is valueless so far as known. The ozone does not enter the blood, and therefore under no circumstances can it reach disease germs in the organs and tissues. In the second place, the highest concentration of ozone that can be tolerated by man has been shown to have no germicidal action, even on bacteria free in the air.

It is theoretically possible that some beneficial action of ozone in disease might arise from a general tonic action on the nervous system and on general metabolism through the ozone irritation of the sensory nerve endings of the respiratory tract. Such favorable action is generally assumed by the champions of the ozone treatment. But when we examine the evidence we find that the most careful experiments on man and animals by Bohr and Maar, and by Hill and Flack, scientists of international standing, *show that ozone in sufficient concentration to have a measurable physiologic action, depresses body metabolism and the respiratory exchange.* All endeavors to demonstrate a primary stimulative action on metabolism have so far failed.

The experiment with smoked paper alluded to by some of your correspondents, was merely indicated as a test that anyone with an ozone generator at hand might perform to disprove the claim (made by some reputable concerns) that "ozone destroys smoke." The statement that "ozone destroys smoke" seems to us as "foolish" and "absurd" as it can seem to anyone. As we showed, its "foolishness" is easily demonstrable.

The masking or suppression, without destruction of the odorous substance, of one odor by another odor is a well known phenomenon in the physiology of the olfactory sense, and is not peculiar to ozone. The smell of ozone itself is not "pure, refreshing, and invigorating," to all persons. It is well known that an irritating effect from ozone is sometimes observed when this gas is used in practical ventilation. A specific instance of the failure of ozone as an "air-purifier" is mentioned in the *Journal* of the American Medical Association, Dec. 20, 1913, p. 2260.

Those who will take the trouble of reading our report cannot fail to note that our results are essentially the same as those obtained by numerous investigators in his country and in Europe. It seems clearly established that ozone in ventilation does not destroy bacteria, has no action on moisture, temperature, dust, or soot, and does not to any appreciable extent destroy odors, although it may mask them. In other words, ozone in ventilation does not make the air pure in any sense. It has not yet been shown that the concentration of ozone which can obtain in ventilation has any directly favorable or unfavorable action on man or animals. But one fact seems significant, namely, that as soon as we reach a concentration of ozone that has a demonstrable physiologic action on man, this action is definitely injurious.

EDWIN O. JORDAN.
A. J. CARLSON.

Chicago, Ill., May 6, 1914.

*"Journal," Amer. Med. Assoc., Sept. 27, 1913; "Eng. News," Nov. 27, 1913.

Comparative Designs of River Steamship Piers, Philadelphia, Penn.

The department of Docks, Wharves & Ferries of the city of Philadelphia, has recently received bids for two steamship piers to be located in the Delaware River, in the new Southwark improvement. On account of radical local differences of opinion as to the relative economy of various types of design, the department made five different designs, three for one pier and two for the other, all of the designs being for the same dimensions and loadings. Bids were received on all five designs and in addition one bidder submitted a sixth design. The details of the designs are given in the accompanying cut and the bid figures are in the accompanying table. The department also comments upon the designs and bids as noted below.

George W. Norris is Director and John Meigs, Assistant Director of the Department.

COMMENTS BY DEPARTMENT OF WHARVES, DOCKS AND FERRIES

The accompanying prices are for the construction of the substructures only, and do not include any portion of the sheds. All of the pier types mentioned are similar in general dimensions, in height of deck (12 ft. above mean low water), and in the superposed loads for which they are designed. The main deck slab is designed for a superposed load of 600 lb. per sq.ft., and the column bases for a second floor superposed load of 300 lb. per sq.ft. and a roof total load of 70 lb. per sq.ft.

In comparing the prices obtained under schedules "A," "B" and "C," in November, 1913, with those under schedules "D" and "E," in March, 1914, the following differences in conditions should be taken into consideration: In the "A," "B" and "C" bids the deck paving was included. The price on this in the low bid was about $18,000, or $0.18 per sq.ft. of gross area of the pier. This should be added to the square-foot price of bids "D" and "E" to make a proper comparison. Also, owing to a change in the specification requirements for piles between the first and second biddings, changing from 15-in. diameter 4 ft. from the butt in the first, to 14-in. diameter 2 ft. from the butt in the latter, an estimated allowance of $10,000, or $0.10 per sq.ft. of pier area, should be added to the square-foot prices of bids "D" and "E" to make a proper comparison.

The quotation for the "C" type includes $25,000 for riprap. This was to be deposited at the toe of the sheet piling and was included because it is an essential portion of the design. The prices for the other different types do not include cobble or gravel fill. It is estimated that there will probably be $10,000 expended for this material to give lateral support to the piles. This costs per square foot of pier area for all types but "C," approximately $0.10.

The prices under Alternate Bids are the quotations of the Raymond Concrete Pile Co. and were used in determining the minimum and maximum costs.

Following is a brief description of each type:

Type "A" consists of: A double row of piles in transverse bays, on 20-ft. centers longitudinally, cut off in a plane approximately 1.5 ft. above low water. The piles are clamped and capped, and this framing covered with a light timber deck, upon which are set concrete walls approximately 11 ft. high. These support a system of reinforced-concrete floor-beams and slabs. On the slabs the deck paving is laid.

Type "B" consists of: Timber piling driven on 5-ft. centers longitudinally and transversely, cut off approximately 1.5 ft. above mean low water, then clamped and capped, and upon the caps a heavy decking placed, which forms a timber platform covering the whole pier area at an elevation of approximately 3 ft. above mean low water. On the outer edges of this decking, along the sides and ends of the pier, concrete walls are constructed approximately 11 ft. in height. A dry fill is deposited on the platform, retained by these concrete walls, and brought to the subgrade of the paving. Upon this fill, a 6-in. concrete base is placed, on which the paving is laid.

Type "C" consists of: A solid earth fill retained by bulkhead walls on the sides and outshore end of the pier. For these bulkheads, piles are driven approximately 5 ft. on centers in both directions, clamped and capped, upon which a heavy timber platform, approximately 25 ft. wide, is laid. Its surface is 3 ft. above mean low water. Timber sheet-piling is driven along the inner edge of this platform and a concrete wall approximately 11 ft. high constructed on the outer end of it. This construction is essentially a bulkhead type and practically consists merely of the standard bulkhead section as used at the ends of the docks, extended around the three sides of the pier. Transverse reinforced-concrete ties connect the longitudinal bulkhead wall together. A wet or dry fill is deposited behind the sheet-piling and on the platform and brought to the underside of the 6-in. concrete slab which supports the paving.

Alternate Bid (Raymond Concrete Pile Co.) consists of: A solid earth fill retained by reinforced-concrete sheet-piling 20 in. thick. This sheet-piling is driven 13 ft. back from the sides of the pier. The design of the column footings inside of the rows of sheet-piling is similar to those of Type "D"—that is, concrete pedestals supported on timber piling. The center rows of column bases outside of the sheeting are supported on concrete pedestals, each one carried by five reinforced piles 18 in. square. Transverse reinforced-concrete beams 12x18 in., running from side to side of the pier, on 20-ft. centers, tie the construction together. The wall and sheeting across the outshore end of the pier are held in place by reinforced-concrete ties running to concrete blocks, supported and braced by vertical and inclined timber piles.

Type "D" consists of: Timber piling in transverse bays, spaced 10 ft. c. to c., cut off and clamped at about 12 ft. above mean low water. Upon these clamps a 10-in. reinforced-concrete floor slab is constructed. On this slab the deck paving is laid. In addition to the above clamps a lower set is also provided at about 2 ft. above mean low water and the piles are thoroughly braced together both longitudinally and transversely.

Type "E" consists of: Piles driven in clusters about 20 ft. apart c. to c., in transverse bays, spaced on 20-ft. centers. These piles are clamped and decked over at about 2.5 ft. above mean low water to support concrete pedestals. These pedestals are approximately 11 ft. high and from them spring the girders of a reinforced-concrete floor system of beams and slabs. On the surface of the slabs the deck paving is laid.

COST DATA FOR PIERS NOS. 38 AND 40, SOUTH DELAWARE WHARVES, PHILADELPHIA (SOUTHWARK IMPROVEMENT)

From bids received in November, 1913, and March, 1914

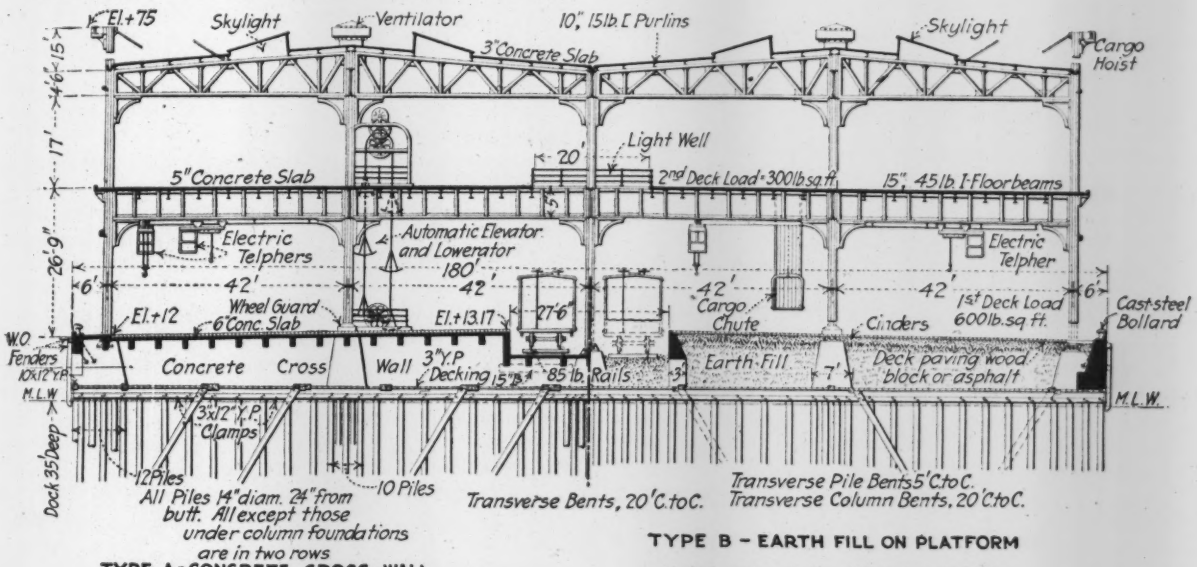
Types of construction. (For description see text)	"A"	"B"	"C"	Alternate	"D"	"E"
Dimensions	Pier 40 551x180 ft. 99,200 sq.ft.	Pier 40 551x180 ft. 99,200 sq.ft.	Pier 40 551x180 ft. 99,200 sq.ft.	Pier 40 551x180 ft. 99,200 sq.ft.	Pier 38 551x180 ft. bulkhead sheds 306x34 ft. 109,700 sq.ft.	Pier 38 551x180 ft. bulkhead sheds 306x34 ft. 109,700 sq.ft.
Low bids	$276,000	$296,000	$300,000	$319,960	$235,730	$245,200
High bids	359,000	340,500	355,000		279,000	290,000
Cost per sq.ft. under low bid	2.77	2.97	3.02	3.225	2.15	2.23
Cost per sq.ft. under high bid	3.63	3.43	3.58		2.54	2.63

UNIT PRICES BID ON PRINCIPAL ITEMS OF WORK

(All prices are for materials in place at the work)

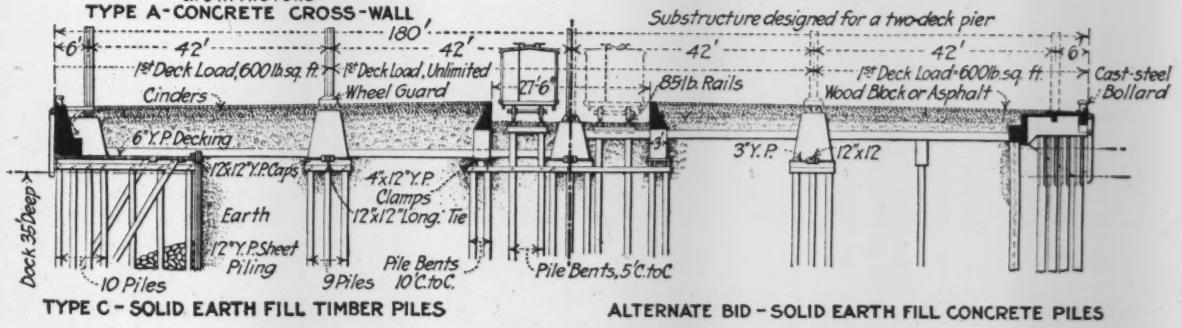
Types.	"A"	"B"	"C"	Alter.	"D"	"E"
Gravel or cobble filling, per ton	$0.65 to $0.80	$0.65 to $0.80	$0.65 to $0.80		$0.50 to $0.75	$0.50 to $0.75
Rip-rap, per ton	1.50 to 2.25	1.50 to 2.25	1.50 to 2.25	$1.25		
Piles	16.00 to 20.00	16.00 to 20.00	16.00 to 20.00	21.00	14.00 to 20.00	14.00 to 20.00
Spur piles	18.00 to 33.55	18.00 to 33.55	18.00 to 33.55	23.80	15.00 to 25.00	15.00 to 25.00
12-in. sheet piling, per ver. lin.ft.	0.70 to 1.25	0.70 to 1.25	0.70 to 1.25	0.80		
Timber—clamps, caps, wales, decking	47.00 to 70.20	47.00 to 70.20	47.00 to 70.20	65.00	52.00 to 70.00	52.00 to 70.00
White oak fenders, per 1000 ft. b.m.	70.00 to 98.40	70.00 to 98.40	70.00 to 98.40	70.00	65.00 to 87.00	65.00 to 87.00
No. 1 reinforced concrete, per cu.yd.	11.00 to 20.00	11.00 to 20.00	11.00 to 20.00	11.00	11.00 to 15.00	11.00 to 15.00
No. 2 mass concrete, per cu.yd.	6.50 to 11.05	6.50 to 11.05	6.50 to 11.05	10.00	7.00 to 10.00	7.00 to 10.00
Reinforcement, per ton	45.00 to 100.00	45.00 to 100.00	45.00 to 100.00	70.00	40.00 to 100.00	40.00 to 100.00
Structural steel, per ton	50.00 to 100.00	50.00 to 100.00	50.00 to 100.00	80.00	55.00 to 100.00	55.00 to 100.00

(The above prices are the minimum and maximum quotations received).



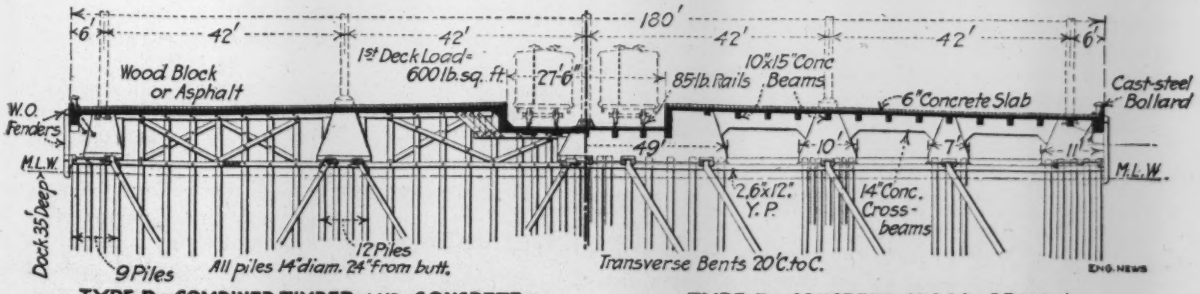
TYPE A - CONCRETE CROSS-WALL

TYPE B - EARTH FILL ON PLATFORM



TYPE C - SOLID EARTH FILL TIMBER PILES

ALTERNATE BID - SOLID EARTH FILL CONCRETE PILES



TYPE D - COMBINED TIMBER AND CONCRETE

TYPE E - CONCRETE CROSS-BEAM

REMARKS AND CONCLUSIONS—The ruling considerations in these designs were: first, practical permanency of construction, and second, as great a degree of economy as was consistent with permanency and stability. No marine borers of any type are prevalent in the waters of the Delaware River as far up as Philadelphia, and, consequently, no necessity existed for providing in these designs against their attacks. None of these designs are suitable for waters infested with this form of marine life.

An average tidal variation of about 6 ft. occurs at Philadelphia, and timber is considered a practically permanent material, as far as decay is concerned, up to the level of about half tide, or say, 3 ft. above mean low water. This elevation is, therefore, made the plane of demarcation between the timber and the concrete in these piers, the former being used up to this level, and the latter above it.

All of these designs are considered to be of permanent character, except Type "D." In this the timber piles extend up to the bottom of the main deck slab, at elevation approximately 12 ft. above mean low water, and their upper ends would be subject to complete renewal in from ten to fifteen years. The portion of the structure subject to decay is readily renewable in this type, and it was thought originally that a material saving in first cost might be accomplished by this design sufficient to offset its partially temporary character. The difference in bid prices obtained, however, was not sufficient to justify the adoption of this type in the contract award.

Of the four permanent types, the "E" design, or the so called concrete-beam type, is the most economical for the particular width of pier under consideration, and presumably for narrower ones. A comparison of the unit prices named, indicates that this type will continue to be the most economical for piers up to 200 ft. in width. For widths above this, the "C" design, or the solid earth-fill type, would be cheaper under local conditions in Philadelphia, its economy over the other designs increasing steadily with the width of the pier.

The Department of Wharves, Docks and Ferries has adopted a policy of wide-pier construction for city wharves, it being believed that structures of upwards of 300 ft. in width are necessary to properly accommodate the handling and storage of inbound and outbound cargo of large, modern ocean freight-carriers, so that the solid earth-fill type will probably be generally used for future municipal steamship piers.

Slides on the Sag Drainage Canal, Chicago*

The Niagara limestone formation which underlies almost the entire vicinity of Chicago, is covered with a layer of drift. This consists generally of a bluish clay in which is embedded stone ranging from fine gravel to large boulders, but for the most part the pieces of rock are less than 1 ft. diameter, with boulders 2 to 3 ft. diameter of rather frequent occurrence.

The usual formation of the drift as encountered in the Sag Valley, where the Calumet-Sag channel is being constructed, is a shallow layer (not exceeding 5 ft.) of a yellow clay, just below the top soil, below which is found the blue clay forming the body of the drift. This blue clay sometimes becomes very dense and is excavated with difficulty. Below this is often encountered a formation of a hard brown clay, sometimes containing gravel and small boulders partially cemented. This material is extremely

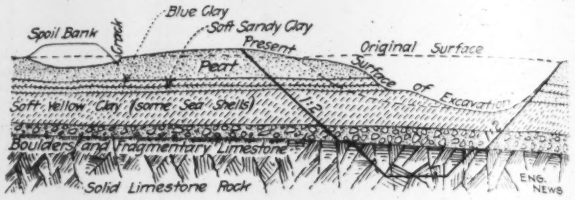


FIG. 1. TYPICAL SECTION OF SLIDING GROUND ON THE CALUMET-SAG DRAINAGE CHANNEL, CHICAGO

difficult of excavation, and often necessitates blasting. Occasionally, a water-bearing layer of blue clay containing a large percentage of sand and silt, and varying in thickness from 1 to 5 ft., is encountered. This silty clay (locally termed "bull-liver") is very unstable and when exposed has a tendency to flow. Its supporting power is, of course, very small, and this formation has constituted one source of trouble in maintaining the canal banks in a state of equilibrium.

Another source of difficulty has been found where seams of clay resembling shale have been encountered. This clay tends to disintegrate and crumble upon exposure, and consequently the bank becomes unsupported and has a tendency to move into the channel. In the swampy regions, of which there is a large area along the Calumet-Sag channel, extensive peat beds are found. This formation sometimes occurs at a depth of 10 to 20 ft., although usually it constitutes the top soil. When relieved of its moisture, it is very light in weight and incapable of supporting a high bank. As a result, considerable difficulty has been encountered in designing the canal slopes and disposing of the spoil to meet this condition.

Fig. 1 shows a typical section of sliding ground on the Calumet-Sag channel, while Fig. 2 shows two views of sliding banks, illustrating the way in which the ground cracks and moves.

Three types of sliding ground have been encountered: (1) Structural breaks resulting from inability of a layer of drift to hold the weight of the bank above it; (2) normal or gravity slides; (3) surface erosion.

The structural breaks are in many respects similar to those encountered at Culebra, on the Panama Canal. They occur at points where a layer of shale disintegrates and crumbles upon exposure to the atmosphere. A crack or fissure then develops in the bank, sometimes at a distance of 200 to 300 ft. from the center line of the channel, and this crack gradually widens and deepens as the bank moves slowly into the channel. Instead of a layer of shale, the prime cause may be a peat stratum, or a soft, silty or unstable clay.

The normal or gravity slide results from the movement of the bank upon a slippery layer of clay or other material, the line of stratification of which is clearly defined. It is due almost entirely to an excavated slope steeper than the angle of repose of the particular formation then

*Extract from the report of a committee of the Sanitary District of Chicago, which investigated the slides on the Panama Canal and bank protection on the Mississippi River, in relation to the troubles encountered on the Sag or Calumet canal from the Calumet River to the Chicago main drainage canal. The committee was composed of three trustees; E. J. Kelly, Assistant Chief Engineer, and Philip Harrington, Resident Engineer.

being excavated, and in many cases it has been further aggravated by superimposing heavy spoil banks on the berm, at a comparatively small distance from the slope.

The sliding, due to surface erosion, is found in many places on both the North Shore channel and the Calumet-Sag channel, and is marked by a gradual sloughing of the surface of the slopes due to the weathering action of the elements. In addition to rain and wave action, the freezing and thawing of the bank tends to further aggravate the sloughing, so that the sowing of shallow-rooted grasses and like vegetation is not in all cases a sufficient preventive means.

Where slides occur from the failure of an underlying stratum to hold up the unbalanced pressure due to the cutting of the channel, or where the bank created by the same operation shows movement upon an underlying stratum of a different material, the only remedy which can be advised is the flattening of the slopes until sufficient weight has been removed to establish equilibrium; in other words, until the angle of repose for the material then moving has been attained. For this reason, it has been maintained that waste or spoil banks should be placed at a considerable distance from the banks in all cases where preliminary borings show indications of formations of this character, and the side slopes of the channel designed flat enough to prevent movement of the bank.

The distance that the spoil bank should be placed from the channel, and the angle of inclination of the side slopes, can be arrived at with any degree of accuracy only by long experience. However, it may be said that the angle of inclination of a slope to prevent movement is in most cases steeper than the angle necessary to retard and stop the movement of a bank which has commenced to slide. For this reason, it is far more economical to excavate the side slopes to the necessary inclination as the excavation is carried down to grade, than it is to allow the material to slide and excavate it afterwards. Not only does the latter method tend to increase the total yardage in any cross-section, but it also involves other difficulties such as extricating machinery, track, etc., with the accompanying loss and expense, and leaves a canal with an irregular bank which invites the rain and weather to commence a process of erosion.

The Calumet-Sag channel, having a cut of about 40 ft., was designed with slopes in earth of 1 on 2, and the spoil banks 50 to 100 ft. from the top of the slope. In some cases this has proved ample, while in others, slides have occurred at points where the material is soft and of unstable character. These conditions have been recognized and much of the later work slopes 1 on 3, with the spoil banks 400 ft. from the top of the slope. In one case, where the material is exceptionally soft, the contractor has been ordered to place no spoil on the berms, but to transport it about ½ mile to a specially provided dump. The policy of the Sanitary District differs from that at Panama in that it is deemed more economical to prevent the material sliding than to let it slide and dig it out. The remedy which has been found most successful for slides due to erosion has been to riprap the side slopes with large-sized rubble in a layer 8 to 10 in. thick, placed upon a 12-in. layer of crushed stone or gravel. The

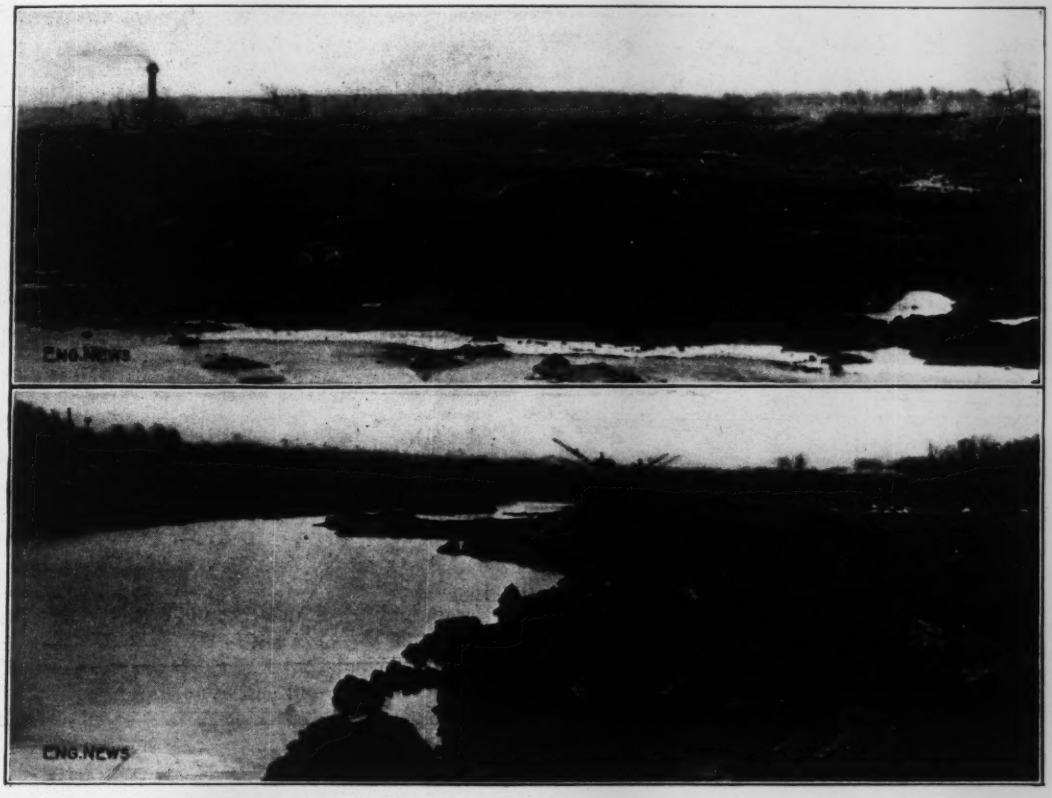


FIG. 2. TYPICAL SLIDING BANKS ON THE CALUMET-SAG DRAINAGE CHANNEL, CHICAGO

latter forms a cushion between the rubble and natural bank and at the same time provides a course for the surface water which would otherwise flow down the slope. It also tends to prevent the frost making any great headway into the slopes. The lower section in Fig. 3 shows this form of paving, and Fig. 4 shows the work in progress on the channel.

In some cases, it has been found advantageous to drive a pile and timber foundation, with batter piles in front, to act as a brace for this riprap-slope pavement. This

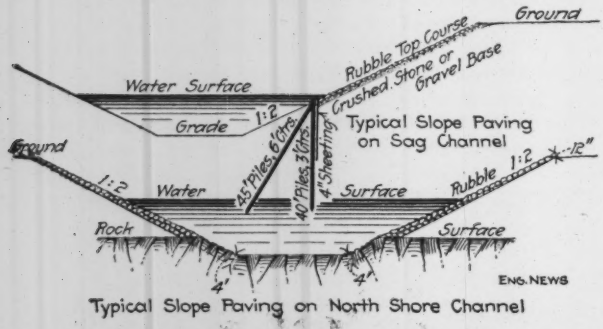


FIG. 3. TYPICAL METHODS OF SLOPE PAVING ON THE CALUMET-SAG AND NORTH-SHORE DRAINAGE CHANNELS, CHICAGO

foundation serves a two-fold purpose: it not only acts as a support for the riprap, but it also tends to prevent further movement of the banks due to the overhead weight. It has been tried on the North Shore channel, and, so far, has proved successful. This is shown in the upper section in Fig. 3.

FIG. 4. PAVING THE SLOPES OF THE CALUMET-SAG DRAINAGE CHANNEL, CHICAGO

However, where riprap can be placed before the water has been turned into the channel, it is better and more economical to carry the slope paving to a solid natural foundation, such as rock or very hard earth. This latter method is now being employed on practically the entire length of the Calumet-Sag channel, the rock necessary for the rubble being excavated from the canal. On the basis of durability, rubble is favored as compared with concrete. On the Sanitary District work, it is not practicable to acquire a 10-mile right-of-way, as on the Panama Canal, so that it is necessary to place the excavated material on the berms of the canal. For this reason, means must be taken, in many cases, to prevent movement of the bank, instead of letting the material slide and afterwards excavating it. Methods adopted by others, and opinions rendered by engineers accustomed to handling slides and slide conditions, have confirmed the judgment of the Engineering Department in advocating the methods now being used by the Sanitary District.

Report on a Sewerage Plan for Cincinnati

That Cincinnati needed a sewerage plan when the studies noted below were begun is shown by the fact that the studies figuratively unearthed 94 miles of sewers not listed in the city records, and by the further facts that the city had no accurate maps and that it had never worked out any scheme for intercepting sewers to clean up streams within the city nor looked ahead to the time when sewage treatment might become necessary.

The report is dated Dec. 27, 1913. It consists of a brief introduction by H. M. Waite, then Chief Engineer Department of Public Works, transmitting to Victor T. Price, Director Public Service, a joint report by H. S. Morse, Engineer-in-Charge, and Harrison P. Eddy, Consulting Engineer of Sewerage Investigations.

The scope of the investigations is summarized in the Morse and Eddy report as follows:

> Upon completion of these preliminary examinations, the scope of the work was determined and an organization effected covering the following branches of the problem: (1) Underground Survey, to obtain records of existing sewers; (2) Topographic Survey, to make a topographic survey and map if the city; (3) New and Relief Sewers, to compute the capacities of existing sewers and design the necessary sewers to relieve congested sewers and to plan new sewerage where required; (4) Intercepting Sewers and Creek Improvements, to plan sewers to collect the sewage now discharged into Duck Creek, Mill Creek, and the Ohio River, and to convey it to proper disposal points, and to prepare plans for the further improvement of these creeks; and (5) Sewage Disposal, to make studies and plans covering the problem of the ultimate disposal of all sewage from the city.

THE UNDERGROUND SURVEY included the "location and plotting of all surface and subsurface structures in the city between property." The plats are 23x32 in. within border lines, 40 ft. to the inch in the downtown district, and 50 ft. to the inch elsewhere. Up to Dec. 27, 1913, there had been plotted 544 sheets covering 386 miles of sewers and other underground structures. There will be about 725 sheets in all.

THE TOPOGRAPHIC SURVEY was begun in May, 1912, and both field and office work were completed late in December, 1913, covering 102.16 sq.mi., of which about 30 sq.mi. were outside the city limits. Under this head the engineers state:

> The triangulation net used in building up the map is tied into the United States trans-continental net along the 39th parallel of latitude, and is of sufficient accuracy for use in a cadastral or property line survey. In this connection we desire to call particular attention to the condition of Cincinnati records as to property corners and surveys. Information of this sort is either in the hands of a few private surveyors, and, therefore, unofficial in character, or is lost, and unless a special effort is made to preserve them, within a few years such records and important landmarks and corners, will have passed out of existence. Therefore, at as early a date as possible a cadastral survey of Cincinnati should be made particularly in the downtown section. Such a survey can be based on the triangulation net of this Topographic Survey and the positions of corners at all street intersections can be accurately determined and properly monumented.

NEW AND RELIEF SEWERS have been designed for nine districts and at the date of the report were in progress for five more districts. The engineers very properly urge that the studies be continued until the whole city has been covered.

THREE INTERCEPTING SEWERS will be required for Cincinnati: Mill Creek, Duck Creek and the Ohio River. The estimates for these are $2,185,000 and $492,000, respectively, for the first two, and from $800,000 to $5,600,000 for the Ohio River interceptor—depending upon the length of the latter, which in turn will depend in part upon the policy adopted as regards sewage treatment.

At the close of December, 1913, work on the contract drawings for the Mill Creek and Duck Creek interceptors was progressing rapidly, and one contract on each had already been let.

SEWAGE DISPOSAL

The population in the Cincinnati Sewerage district is now thought to be over 500,000, and it is estimated that 40 years hence it will be 900,000 in district and 720,000 in Cincinnati. Besides a population of 1,600,000 above Cincinnati discharges sewage into the river and the stream also receives "vast quantities of industrial wastes." The report says:

A consideration of the vast population discharging sewage and wastes into the river may give a wrong impression of the character of the water unless the enormous quantity of water flowing in the river is also borne in mind. The dilution afforded the sewage and other wastes is so great that as the water of the river reaches Cincinnati there are no visible evidences of contamination, and it is only by the most delicate bacterial tests that satisfactory and conclusive evidence of contamination is obtained.

Recognizing the fact that the Ohio River water was unsatisfactory and unsafe for domestic purposes, Cincinnati at great expense has installed a water treatment plant. A study of the accurate and detailed records which have been kept since it was put into operation shows conclusively that the plant is capable of removing the dangerous and undesirable elements from the water, and that it is turning out a water which is safe for domestic consumption.

Even more convincing evidence on this subject is obtained from the typhoid case and death rate statistics, which show that while prior to the introduction of filtered water, Cincinnati had a death rate from typhoid higher than the average rate of other large cities in the country, since the introduction of filtered water the typhoid death rate has been materially lower than this average. As typhoid fever is well known to be a water-borne disease, and as it is one which is commonly prevalent, these facts show conclusively that the water treatment plant is capable of producing from raw Ohio River water a water which is safe for domestic consumption.

It is difficult to estimate with accuracy the length of time required for water to pass from Cincinnati to Louisville—a distance of 133 miles by way of the river. Such estimates as have been made, however, indicate that the probable time of passage varies from a minimum of one day to a maximum, under natural conditions of flow, of perhaps four days. As the typhoid bacillus is capable of living in such waters for several days and possibly for several weeks, there is little doubt that typhoid germs discharged at Cincinnati may reach Louisville while still possessed of their normal vitality. It is, therefore, important to know about the quality of water at Louisville, as it has received, in addition to the sewage discharged into it above Cincinnati, that contributed by the Cincinnati district, and by some smaller cities and towns below Cincinnati.

Louisville is provided with a water treatment plant similar to that installed at Cincinnati. A study of the results of treatment at Louisville shows conclusively that this plant is capable of producing a satisfactory and safe water from the raw water of the Ohio River. While the data available for cities and towns between Cincinnati and Louisville are not complete and satisfactory, there appears to be sufficient evidence to demonstrate with certainty that well designed and carefully operated water-treatment plants are capable of rendering raw water, taken from the Ohio River at any of these cities or towns, safe and satisfactory for domestic consumption.

From the studies made, it appears safe to conclude that Ohio River water at Cincinnati and between Cincinnati and Louisville is not so contaminated as to render it unsuitable for domestic supplies when treated by a well designed and carefully managed water-treatment plant; and furthermore, there appears to be no evidence that the contamination of this water is so great as to place a load upon such a plant in excess of that which would be placed upon it were the sewage of Cincinnati excluded from the river, or treated by the recognized methods of sewage treatment before its discharge. It is, however, absolutely essential that all water-supplies drawn from the Ohio River should be treated before being distributed to consumers.

The natural condition of the river will be somewhat modified by the completion of the Government project to create a 9-ft. navigation stage between Pittsburgh and Cairo. One of the dams included in this project is located at Fernbank, which is about twelve miles below the center of the city. This dam holds back the water, forming a pool about 21 miles in length. When the whole project is completed, the Ohio River at times of extreme low flow will consist practically of a series of pools, through which the water will flow at relatively low velocity. Considerable attention has been given to the probable effect of this condition upon the self-purification of the river. The evidence thus far accumulated, while not as complete as is desirable, because of the limited time, and particularly because of the relatively high discharge during 1912, indicates that the river will be in a better condition to carry on the processes of self-purification after the dams are built and used than in its natural condition. This is due to conditions producing a better mixing of the sewage with the river water, to sedimentation and to the better opportunities for the growth of aquatic plants.

After reviewing the various methods of sewage treatment now in use and their availability to meet possible future conditions at Cincinnati, this portion of the report concludes:

In view of the fact that it does not appear that sewage treatment will be necessary at Cincinnati for a number of years, it does not seem wise to recommend any specific method of treatment or that any site be secured for a treatment plant. Methods of sewage treatment have been improved, and new methods have been devised during recent years, and much further improvement may reasonably be expected during years to come. It would, therefore, seem idle to take steps at this time which would commit the city of Cincinnati in any way to a particular method of treatment. It is necessary and wise, however, to so plan and lay out the system of intercepting sewers which may be built from time to time that they may be brought together in harmony with any method for the treatment of sewage which may be adopted; and should it later prove advisable to intercept the sewage along the Ohio River, before a sewage treatment plant is needed, the site on which such a plant is to be built should then be selected so that the intercepter may be laid out with a view to conveying sewage to that site.

From these investigations, the conclusion is that Cincinnati is not at present justified in going to the expense of building and operating a sewage-treatment plant, since the benefit derived therefrom would not be commensurate with the expense. It appears to be entirely practicable to so treat the Ohio River water as to render it safe and satisfactory for domestic supplies below the city of Cincinnati, as well as above it. A plant for the treatment of a public water-supply is likely to be more skillfully and conscientiously operated than a sewage-treatment plant, because it is easier to obtain funds, not only for construction but for operation, as the community providing the funds is the community benefited by the results.

It is, therefore, recommended that Cincinnati continue the policy of discharging its sewage into the Ohio River, and that particular attention be given to so constructing outlets as to secure a thorough mixing of the sewage with the river water. Particular attention should be given to this point in the design of the outlet structure for the Mill Creek Intercepter now under construction. This outlet should be extended well out into the river, and it is not improbable that it will be desirable to construct a "multiple-outlet" structure so that the sewage discharged through a number of openings may be more completely diffused through the river water.

The sewage discharged by the Mill Creek Intercepter will be inoffensive as compared with that discharged by Mill Creek during periods of extreme low water. It will not make so great immediate demands upon the dissolved oxygen of the river water as the highly putrid water discharged from Mill Creek. The quantity of organic matter will probably be fully as great as that now discharged, but it will be in a condition which will not undergo putrefaction as rapidly, so that time will be available for it to pass a considerable distance below the city and become thoroughly mingled with the river water before there is opportunity for putrefaction. After the sewage is so thoroughly mingled and mixed with the river water, there appears to be little danger of putrefaction in the river, since the quantity of dissolved oxygen brought into intimate contact with the particles of organic matter is so great that the inoffensive process of oxidation takes place in its stead.

Steel Sheet-Pile Coffer-Dam Failure

The failure of the timber bracing of a steel sheet-pile coffer-dam, approximately 30x60 ft. in plan and about 25 ft. deep, at South Norwalk, Conn., May 21, resulted in the death of the contractor's engineer and the foreman carpenter. Two other men who were in the coffer-dam at the time of the collapse escaped with minor injuries. The coffer-dam was built for one of the main piers of a rolling-lift drawbridge span of a new reinforced-concrete arch bridge over the Norwalk River between South Norwalk and East Norwalk. The accident occurred about 11 p.m., shortly after the coffer-dam had been pumped out for the first time, and under a head of water some 3 ft. less than the high tide it had withstood about an hour before, when the contractor, the city engineer and the resident engineer were inside the coffer-dam to inspect it. It is reported that at the time of failure there was 6 ft. of water in the dam, although shortly before it had been practically dry.

The point of the initial failure and the exact cause of the collapse of the coffer-dam bracing will probably never be known, for the two men responsible for its design and construction were both killed in its collapse. The sheeting, 14-in. Lackawanna steel interlocking arch sheet-piling, remained intact, as is shown in the accompanying views. The piling was either bent or tipped over so that the two opposite sides nearly meet in the center of the coffer-dam; but above water there is no rupture in the coffer-dam itself.

The chief features of the design of the framework, sketched by a member of the editorial staff of ENGINEERING NEWS from testimony given at the coroner's inquest, are shown in Fig. 2. The 12x12-in. transverse struts and posts were fastened to the waling and center longitudinal struts by short 3-in. plank fish-plates bolted through the timbers. There were also longitudinal diagonal braces both through the center and the side frames, made of 3x10-in. timbers. The timbers were of ordinary spruce and pine lumber, purchased new at the time the frame was built last winter.

The coffer-dam frame was built as a unit during the winter and towed into place; and most of it had been under water for several months. The bottom of the river was previously prepared by dredging away about 6 ft.

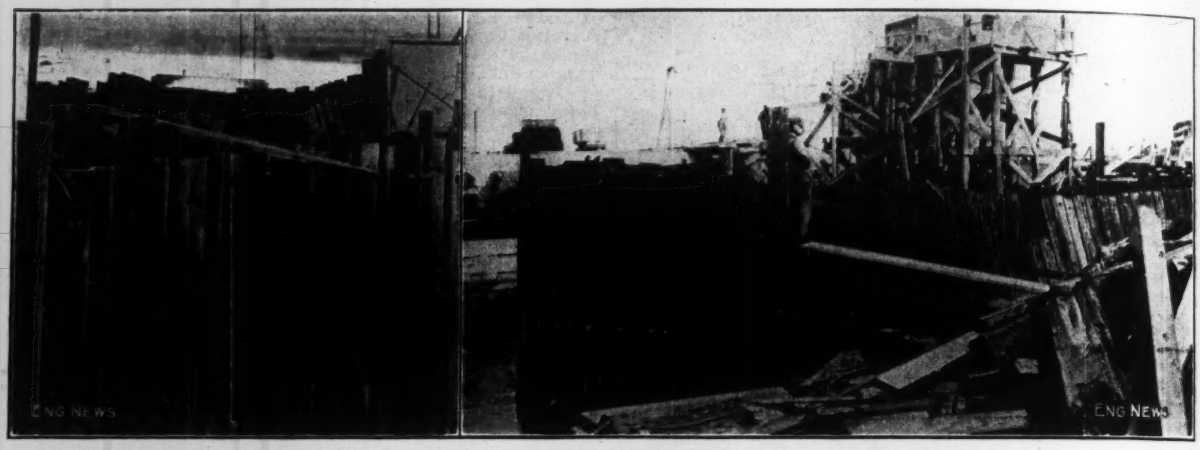


FIG. 1. VIEWS OF STEEL SHEET-PILE COFFER-DAM, SOUTH NORWALK, CONN., AFTER COLLAPSE OF TIMBER FRAMING, MAY 21.

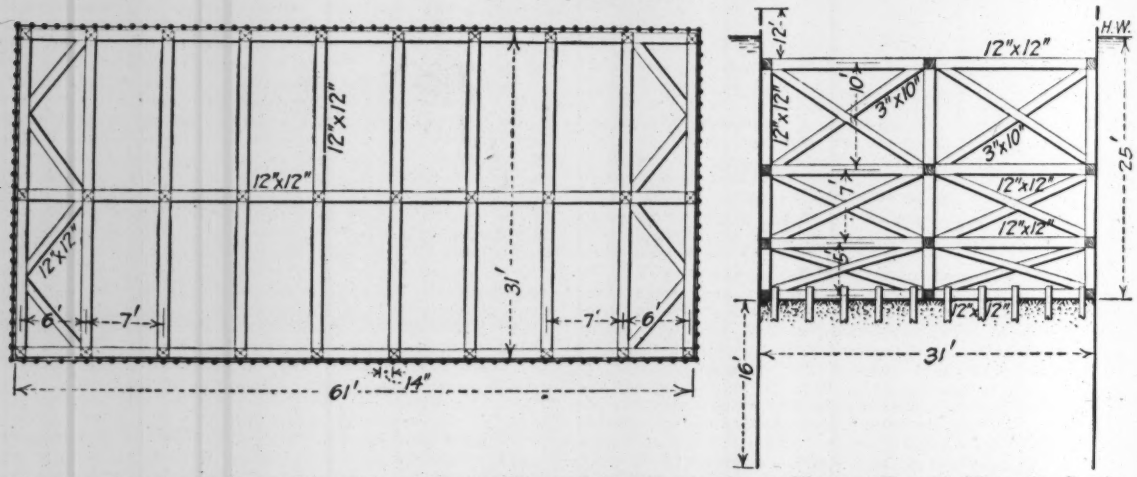


FIG. 2. PLAN AND CROSS-SECTION OF STEEL SHEET-PILE COFFER-DAM, WHICH FAILED MAY 21, SOUTH NORWALK, CONN.

of mud and silt and driving wood bearing piles. The frame was then placed, the lowest waling being fastened to the foundation piles, and 50-ft. interlocking steel sheet-piles driven so as to have a footing of about 16 ft. in the bottom, which, under the silt, was supposed to be a hard gravel.

Considerable difficulty was experienced in pumping out the coffer-dam (preparatory to placing the concrete footing) with two centrifugal pumps, so there were recently added two more, making four pumps with a combined capacity of 8000 cu.ft. per min. Three were electrically driven and one steam operated. The pumps and operating switches were mounted on the timber bracing within the coffer-dam near the longitudinal bent. As the water was lowered, special precaution was taken, it was stated, to drive wedges around the waling frames wherever the sheet-piling had sprung away. It is reported, however, that the dam was not tight but leaked considerably during the pumping process

It is proposed to spring the coffer-dam walls back into place by hauling lines and jacks. The bottom had not been examined at the time this article goes to press, so that it is impossible to state whether the bottom waling frame remained intact or not.

The contractor for the bridge and coffer-dam is the McHarg-Barton Co., of New York City; the work is under the general supervision of C. L. Barton, member of the firm, and the coffer-dam design and construction was under the immediate supervision of R. C. Dewar, the contractor's engineer who was killed. The consulting engineer for the city is John E. Greiner of Baltimore, Md., for whom W. Watters Pagon is Resident Engineer.

State Supervision of Dams in Connecticut

The State of Connecticut has for many years been at the head of the various commonwealths in controlling the construction and operation of dams. In 1878 a law was passed regulating dam and reservoir construction. This law provided that the member of the Board of Railroad Commissioners who was a civil engineer and one civil engineer residing in each congressional district in the state should constitute a board of civil engineers and have the supervision of all dams and reservoirs. This board, acting individually, inspected whatever dams were considered to be of danger, their initiative being taken on the petition of local officers or of two or more persons who felt that they might suffer loss by the breakage of any dam or reservoir.

While this law was far ahead of any similar law in any other state it still had a number of deficiencies. No record was kept of the dams inspected and the inspection was not as mandatory as could have been wished.

During the past year the Connecticut legislature passed an amended law which permitted of a much better organization of the board and much better rules for the inspection of dams. This act, together with rules and information concerning the inspection of dams in the state, has been recently compiled in a booklet which should be obtainable from E. W. Bush, who is the secretary of the State of Connecticut Board of Civil Engineers, at Lyme, Conn. The essentials of the act are as follows:

(1) The commissioner of rivers, harbors and bridges, who is a civil engineer, and one civil engineer residing in each congressional district to be appointed by him, shall constitute a board of civil engineers who shall have supervision of all dams and reservoirs.

(2) The members of this board shall continue in office for two years. The members of the board shall each receive $10 per day and their necessary and reasonable expenses while actually employed.

(3) Upon the application of two or more persons or corporations who would suffer loss or damage by the taking away of any dam or reservoir a member of the board shall forthwith inspect the same and if in his opinion such dam or reservoir is not sufficiently strong to withstand the action of water under any circumstances he shall at once serve notice upon the person or corporation owning or having the care and control of the same to place the dam or reservoir in a safe condition under the supervision of the said engineer. When such repairs are completed to the satisfaction of said engineer he shall issue a certificate of such acceptance to such person or corporation who shall record the same in the land records of the town in which said dam or reservoir is located. If said engineer shall find the dam or the reservoir to be secure and safe, the expense of such inspection shall be paid by the applicant.

(4) This is an entirely new section and reads as follows:

Before any person or corporation shall construct a dam or reservoir, or alter, add to, or replace any dam in a locality where life or property may be endangered through the insufficiency thereof, the plans, specifications and necessary data for such dam or reservoir shall be submitted to a member of said Board of Civil Engineers who shall examine the ground where the dam or reservoir is to be built and the plans and specifications therefor. If he approves the same he shall issue a certificate authorizing the construction of such dam or reservoir or such alterations, additions or replacements. No such dam or reservoir shall be constructed nor such alterations, additions, or replacements made without such approval and certificate. If said state engineer deems it necessary he may before issuing said certificate call in for consultation and advice the State Board of Civil Engineers or any member of said Board.

5) The engineer inspecting shall issue a certificate approving the same, which certificate shall be recorded in the land records of the town.

(6) Any person aggrieved by any decision of the member of the board may appeal to any judge of the superior court within twenty days of the date of decision.

(7) Every person or corporation who shall build or repair any dam or reservoir except in compliance with the provisions of the act, or shall use any dam or reservoir when constructed or repaired without obtaining the certificate provided shall forfeit $500 for the use of the state. Any person or corporation constructing or repairing a dam or reservoir or using such dam or reservoir without complying with the provisions of this act may be enjoined from constructing or using such dam or reservoir.

(8) The state attorney of the town or county in which such dam or reservoir is located upon the complaint of any engineer designated under the provisions of this act shall institute an action to recover such forfeiture and to enjoin the construction and use of such dam or reservoir.

In addition to the foregoing act, the Board of Civil Engineers appointed have adopted certain rules, which are in brief as follows:

(1) There shall be submitted for preliminary ap-

proval two copies each of the drawings, specifications, description of the proposed work, etc. If approved, one copy of each shall be so marked and returned to the owner, the other copy being retained by the board member for use during construction.

(2) Before obtaining the final certificate of approval, the owner shall for permanent record file with the member of the board issuing the certificate plans on tracing cloth or plans on cloth by the in-ko-graf or other equally permanent and satisfactory process of printing and specifications or written description which show and describe the work as actually constructed. These plans and written data shall be certified as correct by a person who was in responsible charge during construction and who knows to his best knowledge and belief that the facts certified to are correct.

(3) The permanent plans and records shall from time to time be filed with the librarian of the State library. The members of the board while acting in their official capacity shall not render professional services to owners in connection with the dams and reservoirs concerning which they are officially engaged. The consent of four members shall be necessary for action by the board.

There are six members in the Board of Civil Engineers; one member at large and one from each of the five congressional districts of the state. It is customary to make application for approval to the member representing the congressional district in which the dam is located, but the law permits any member of the board to act in any location in the state. The members of the board are Chairman, Wm. H. Cadwell, who is the commissfoner of rivers, harbors and bridges who is a civil engineer, New Britain, Conn.; the secretary is Edward W. Bush, Lyme, Conn., and the other members are W. Burt, Hartford, Albert B. Hill, New Haven, Forest G. Scofield, Bridgeport, and Wm. G. Smith, Waterbury.

NEWS NOTES

A Municipal Garbage Reduction Plant will be built at Detroit, having a maximum capacity of 300 tons per day. The city garbage amounts to some 50,000 tons per annum. Geo. H. Fenkell, Commissioner of Public Works, has secured an appropriation of $475,000 for the project. A site has been purchased and it is proposed to embody in the works in every way the best practice of the day. The plant is to be in operation by July, 1916. Smith, Hinchman & Grylls, of Detroit, have been appointed Engineers to prepare plans and specifications for buildings and equipment.

A Break in a 16-in. Water Main at Youngstown, Ohio, some months ago, appears to have been caused by the weight of a 36-in. main above it. There was supposed to be a 3-in. clearance between the two mains, but after a break and washout it was found that the large main was resting on the small one. Apparently the clearance was not maintained during construction or else the 36-in. main was not adequately supported, so its weight came on the 16-in. main and broke it. There had been no previous break in the 16-in. main. J. S. Lewis is Superintendent of Water-Works; Veeder Heasley is Director of Public Works, and F. M. Lillie is City Engineer of Youngstown.

Submarine Signal Bells as equipment for ship lifeboats are suggested by the experience of a lifeboat of the burned transatlantic steamer "Columbian." This boat drifted for two weeks directly in the path of transatlantic liners, and her occupants saw several steamers, but were unable to signal them. There were fifteen men in the boat, and eleven died before rescue came. Had ships been equipped with submarine signal-receiving apparatus and had the drifting lifeboat of the "Columbian" been equipped with a submarine bell, it could have signaled passing steamers a long distance away. It is stated that the sound from a submarine bell rung by hand can be detected at a distance of several miles.

The North River Bridge and Tunnel Project in New York City is now proposed to be financed by bonding the New Jersey counties adjacent to New York for part of the cost of the structure. The recent New Jersey Legislature passed a law providing for the appointment by New Jersey of a new State Commission, with power to make final preparations and go ahead with construction. A commission is to be appointed by the Governor as soon as at least three of the five counties agree to pay for preliminary expenses up to $10,000 each. The estimated cost of a Hudson River Bridge at 57th St. is $42,000,000, and the cost of two tunnels under the Hudson opposite Canal St., New York City, at about $11,000,000.

Municipal Railway Extensions in San Francisco continue. Proposals for a Stockton St., Columbus Ave. and Fort Mason line will be received by the Board of Public Works June 3. (The tunnel for this line was described in "Engineering News," Feb. 12, 1914). This will run from Market St. north to connect with the present Union St. (municipal) line after crossing the Geary St. (municipal) line. It will leave the former, after a short run, however, branching west to Fort Mason Reservation and connecting there with the Van Ness Ave. (municipal) line not completed. All these roads not running are expected to be in operation Oct. 1. Construction of the Potrero Ave. extension started May 19; this is the first line south of Market St.

Supply of Gas at Less Than Cost has been permitted by the Massachusetts Board of Gas and Electric Light Commissioners for the municipal gas works of Middleborough. The price up to the present has been $1.75 per thousand cubic feet maximum. The Commission finds that these old prices were too high to encourage the use of gas for light, heat or cooking and that if the output can be increased the cost will come down to the uniform rate of $1.50 to be instituted. The management is held to be taking only an ordinary business risk and that temporarily. Through the benefit of trust funds given by the late T. S. Pierce and the sale of water rights to the city of Taunton, new water-gas equipment has been added without increasing the public debt.

More Changes in the Boston, Mass., Engineering Departments—To effect a saving of $19,260 per annum Mayor Curley, of Boston, Mass., has removed seven employees of the Street Laying Out Department and reduced the salaries of 44 other employees of the same department. The Chief Engineer's salary has been reduced from $3500 to $3300 per annum, and the Assistant Chief Engineer's from $2800 to $2600. Two $2500 per annum Assistant Engineers have been removed, and the salaries of several Assistant Engineers have been reduced from $1600 to $1408.50 per annum. Draftsmen have been reduced from $1300 to $1200 per annum; transitmen from $1300 and $1200 to $1100 per annum. Radical salary reductions in the Department of Public Works were noted in our issue of Mar. 12, 1914, p. 589.

Six Tickets for a Quarter were denied the City of Schenectady in its case before the Public Service Commission for the Second District of New York against the Schenectady Railway Co. All the Commissioners concurred and two filed opinions. One by Wm. T. Emmett holds that the issue is whether an unreasonable income is now produced. That is not to be fixed at a point where the company can just avoid insolvency else private capital will be driven out. Particularly is such an unfriendly attitude not to be taken when industrial conditions are not the best. The present net income is shown to be only 8 to 9% on the value placed by the city on the street railway property while the company faces considerable expense in complying with the various orders of the Commission and provisions of the public-service law.

Measurements of Railway Mileage in Massachusetts were recently proposed because of complaints that railways in that state were collecting more transportation mileage than the distances warranted. The State Commissioner of Weights and Measures arranged to measure all the railroads in the state by equipping an automobile with flanged wheels and an odometer and, after rating it on a measured length of track, running it over the various railways, recording the actual distances between stations.

The plan has been abandoned, however, for the Interstate Commerce Commission has notified the state officials that, in connection with the physical valuation of the railroads now in progress, it will measure the railways of Masachusetts within eighteen months and this measurement by the federal government would supersede any measurements the state might make for the adjustment of fares.

A Switch Lever Working Out of Socket Caused the Derailment of a Passenger Train on the Wabash R.R. at Decatur, Ill., on May 5. A report of this accident furnished to A. O. Cunningham, Chief Engineer of the Wabash R.R., stated that

The derailment of No. 1 at Decatur was caused by the switch lever working out of socket, permitting the switch points to open and derail the Pullman sleeper and the Wabash dining car, the two rear cars on the train. Investigation developed the fact that the switch had been used by a yard crew at about 5:30 p.m., an hour and fifteen minutes before the derailment occurred; and it is evident that the switchman had neglected at that time to place the lock in the hasp and to lock it. This permitted the lever to work out of the socket as the forward portion of the train passed over the switch.

The sleeper and the dining car sideswiped a steel coal car standing on an adjacent track, and killed two trespassers. In addition, two passengers in the sleeper, the train conductor, and fire employees of the dining and sleeping cars, were slightly injured.

"Tap-Line" Railroads have been adjudged common carriers by the U. S. Supreme Court in the case of the Louisiana & Pacific Ry. and others. Therefore they are held entitled to the division of through rates denied some time ago by the Interstate Commerce Commission on the grounds that these were industrial branch lines owned and operated by the concerns furnishing their freight.

The Court, Justice Day writing the opinion, defines a common carrier according to the existence of rights of the public to use the facilities rather than extent of business. It was pointed out that while abuses and discriminations existed, yet such were not grounds for condemning the roads as common carriers; it was affirmed to be within the Commission's power to regulate the division of joint rates in every such case.

This decision is expected to have favorable effect on the pending application of railroads for a 5% increase in freight rates. The Commission has already stated that the advance may not be necessary if joint rates are not to be divided with tap lines.

The Bridge Dynamiters Convicted of conspiracy to transport dynamite on passenger trains in 1910 and 1911, and who recently appealed to the United States Circuit Court of Appeals, were ordered (May 16) to report June 6 to resume the serving of their sentences in Fort Leavenworth penitentiary. The explosives transported were used to wreck bridges, buildings and other structures erected by nonunion labor. The men returned to prison were Frank M. Ryan, President of the International Association of Bridge and Structural Iron Workers, and 19 other members of that organization.

Originally, 39 men were found guilty in the United States District Court at Indianapolis, Ind. Six of the 39 received suspended sentences. The other 33 entered the penitentiary on Jan. 1, 1913. Of these, 32 appealed. The United States Circuit Court of Appeals ordered their release on bonds of $10,000 for each year of sentence imposed. Two were paroled. All save six of the remaining 30 were released on bonds. The court granted new trials to six and denied new trials to 24. Sentences of from 1 to 7 years must be served by the 20 men returned to prison.

Track Elevation of the surface extension of the Chicago & Oak Park Elevated R.R. has been ordered by the Illinois Public Utilities Commission. At Laramie Ave. the trains descend to the surface and run along the side of Lake St. close beside the elevated tracks of the Chicago & Northwestern Ry., and have stations at intervals. As the trains cross the ends of the subways under the C. &. N. W. Ry. the situation is very dangerous, and accidents have been numerous. The elevation of the tracks has been a subject of agitation for several years, but the company has always pleaded financial inability. The company is now owned by the Chicago Elevated Railways Co. (which owns all the local electric elevated lines) and is in the hands of receivers. The order of the Commission provides that plans for the work must be filed by July 1, and that work must be commenced within 60 days after approval of the plans. Until the work is completed all trains must stop at every street crossing. It is provided also that if the receivers refuse to carry out this work the company must cease to operate trains on its surface lines.

Pisgah Forest, North Carolina, part of the estate of the late George W. Vanderbilt, is to be purchased by the federal government, the National Forest Reservation Commission having approved the offer of $5 per acre. There are 86,700 acres. The price to be paid is less than the average for other tracts already acquired although this has been developed into one of the finest preserves in the country. It covers the entire eastern slope and portions of the northern and western slopes of the Pisgah range—one of the most prominent in the southern Appalachians.

The lands were offered to the government by Mrs. Edith S. Vanderbilt to insure their perpetuation in entirety and present development, the price of the total area being placed at some $200,000 below that of former negotiations for it as an indirect contribution to insure its purchase by the government.

Existing contracts call for removing certain portions of the merchantable timber stand on some four-fifths of the area, though these contracts require best forestry practice. The forest is already improved with buildings, roads and trails which will assist in the government administration. This brings the purchased and approved tracts in the Eastern mountains up to 5,077,000 acres.

Conservation of Water on a very large scale is contemplated by about thirty-five large manufacturing concerns in the Naugatuck Valley between Torrington and Derby, Conn.

General meetings have been held and a committee consisting of Charles H. Preston, Jr., M. Am. Soc. C. E., Consulting Engineer of Waterbury, Conn., Ex-Senator Lewis A. Platt, president the Platt Bros & Co., of Waterbury, and W. H. H. Wooster, secretary. The Seymour Manufacturing Co., of Seymour, Conn., have been appointed to look into the proposed scheme and report on its feasibility, cost, location, etc.

At a meeting held on May 4, 1914, this committee reported that it would be possible to construct three reservoirs by building three dams at a cost of $1,000,000. The dams would flood 1225 acres and would store 23,571,920,000 gallons of water.

It is thought that each town and city along the entire valley will coöperate in financing this great development, the idea of which is to have a sufficient amount of water on hand at all times of the year for the numerous manufacturing plants and also to flush the Naugatuck River to an entirely sanitary condition. A plan is being developed at the present time by which, if carried out, would produce sufficient funds to carry out the proposition.

The Empire Suspension Bridge, Panama Canal, described in "Engineering News" of Dec. 16, 1909, is being dismantled, after over five years of useful service. The erection of the structure was begun on Mar. 11, 1909, under the supervision of A. S. Zinn, formerly Resident Engineer of the old Central division, and it was formally opened to public traffic on July 31 of the same year. It was the first bridge to be constructed across Culebra cut, and was designed not only to furnish a means of communication between the east and the west sides of the canal, but also to carry the compressed air and water mains over the cut. It was built with a view of being taken down before the opening of canal navigation, as its height above the water line is only 108.5 ft., and, therefore, would not clear the masts of the larger vessels.

During the height of steamshovel work in the cut at Empire, and before the depopulation of the Canal Zone was begun, there was a large amount of traffic over the bridge. It served the Las Cascades plantation road extending toward the eastern boundary of the Zone, and of a number of trails leading from this road into the interior. The removal of the structure will cause a diversion of what remains of this traffic north and south, north as far as Gamboa, when the new Empire-Gamboa road is opened to use, and south to Panama by way of the Canal Zone highway to Paraiso, Pedro Miguel, and Corozal.

New Electric Tunnel Locomotives on the Michigan Central will soon be in service at the Detroit tunnel, pulling trains between Windsor, Ont., and the new terminal in Detroit. Up to the present six locomotives have been in service. These weigh 100 tons, and are capable of hauling a train weighing 900 tons behind engine (as described in "Engineering News," June 24 and July 1, 1909).

Passenger trains are hauled usually by one locomotive, but occasionally two are used. For freight trains, up to three locomotives are employed (one in front, two behind). The average adhesion coefficient developed by the locomotives is about 25%, but as much as 60,000 lb. drawbar pull has been exerted by a single unit under favorable conditions. While the old locomotives can be combined for any tractive effort, somewhat heavier units became desirable, on account of general use of steel cars in passenger trains, and increase in average length and weight of freight trains. Four new locomotives, soon to be delivered, were therefore ordered of 120-ton weight, i.e., 20 tons heavier than the old. Each of the new units will be capable of hauling a train of more than 1000 tons weight. The new locomotives are precisely the same as the old ones, and will be interchangeable with them in all parts, except for more copper in the motors and ballast-weight added in the cab floor. The first of the new locomotives will be delivered in a week or two, the others to follow at intervals of a month.

The Hamburg-American Steamer "Vaterland," which arrived in New York on May 21, required over four hours for docking after her arrival at the pier in Hoboken. There was a strong ebb tide at the time, and when the ship was brought around broadside to the current to enter the slip, a tugboat with five barges got in the way and compelled delay. Before this tow had steamed out of the way, the tide carried the "Vaterland" too far downstream to make the slip. She backed out into the river to avoid the shoal water on the New Jersey side of the river, and floated down over a mile before she was turned back upstream for a new try at the pier. When the "Vaterland," with her 950 ft. of length, lay across the channel, she practically blocked north and south traffic of large vessels on the North River, and the "Barbarossa," of the North German Lloyd line was delayed half an hour in sailing in order to get safely past the great ship.

In leaving her pier on the return voyage, on May 26, the "Vaterland" had an equally exciting time, although the departure was timed for a slack tide. In backing away from the Hoboken berth and before she could be straightened downstream, she drifted over to the Manhattan shore, where her stern stopped just short of a pier of the Southern Pacific Co. The back wash from her propellers caused two steamships lying in the slip beside the pier to break their hawsers and drift against the shore bulkhead. A coal barge in the slip was sunk.

The Indiana Flood Prevention Commission, appointed by Gov. Ralston, as noted in our issue of May 14, has submitted a preliminary report, which includes the following paragraphs:

It is apparent that from lack of sufficient time and means the flood commission should confine its immediate efforts to the following work, the results of which should form a sufficient basis for a preliminary report and drafting of intelligent legislative recommendations:

1. To gather together, before the data has been lost, and while it is fresh in the recollection or in the records, the height of flood waters of March, 1913.
2. To determine the amount of damages resulting from the flood of March, 1913.
3. To summarize the rainfall and runoff for the various watersheds in the state, and to determine the character of existing obstructions and levees in the bed and flood planes of the main streams.
4. To present an account of the history of past floods in Indiana, so that some judgment for recommendation may be had as to the probable future floods that may be provided against.
5. An account of the steps taken by certain committees to protect themselves against future floods, and the steps taken by railroads and county commissioners to build new bridges to provide larger openings.
6. An account of the measure of legislation under way in other states to control floods and provide state-wide legislation by which the flood protection works in each watershed shall supplement each other and form parts of an adequate, unified, mutually consistent and comprehensive system.
7. For the information of the next legislature an account of the methods adopted for controlling floods by construction of reservoirs, by straightening channels, and the best uses of the flood plane of the rivers which may be subjected to periodical overflow.

We recommend, for the securing of this information and its compilation for the commission's consideration, the appointment of a special commission of five to take charge of this work, under the commission's direction, and press it to the promptest conclusion.

We recommend that said special committee be authorized to employ the necessary assistance to perform this work, estmated as follows:

One engineer in charge; one assistant engineer; two assistants; one stenographer.

We recommend the securing of a fund of $10,000 with which to defray the salaries and expenses of these employees and the traveling expenses of the members of the special committee, as we have carefully estimated that the work projected cannot be properly performed with any less amount.

First Operation of the Panama Lock-Control Board (described in "Engineering News," Dec. 25, 1913) was made on May 9, when the entire west flight at Gatun was used. The 1000-ft. chamber was used, as for the largest vessel, though the trial was made without a boat. The lower lock was at sea level at the start, while the middle level was 28.1 ft. The upper lock was then filled to lake level by opening rising steam valves Nos. 260 and 261, each valve being opened half, and then opened to the full. At 10:05 a.m., the lock was filled to lake level, and the upper guard and operating gates were opened to the lake. The lock was then ready for the entrance of the vessel, except that the fender chains, which are not yet actually installed, are to be lowered before a vessel can enter, and again raised before the gates can be closed.

The upper gates were closed at 10:15 a.m., after the hypothetical vessel had entered the lock. The lockage was then carried on step by step, as shown on the accompanying log of operations, just as for a vessel.

9:42. Rising stem valve 260, opened half way.
9:45. Rising stem valve 261, opened half way.
9:59. Rising stem valve 261, opened all the way.
10:01. Rising stem valve 260, opened all the way.
10:05. Gates 40 and 36 opened.
10:05½. Gates 39 and 35 opened.
10:14½. Gates 40 and 36 closed.
10:15. Gates 39 and 35 closed.
10:15. Rising stem valve 260 closed.
10:16. Rising stem valve 261 closed.
10:19. Rising stem valve 249 opened.
10:25. Gate 27 opened.
10:25:30. Gate 28 opened.
10:25:30. Rising stem valve 248 opened.
Middle level equalized.
10:33. Gates 23 and 24 opened.
10:37. Gates 28 and 24 closed.
10:37:30. Gates 27 and 23 closed.
10:42. Rising stem valve 248 closed.
10:43:30. Rising stem valve 249 closed.
10:48. Rising stem valve 226 opened.
10:51. Rising stem valve 227 opened.
11:03. Levels equalized.
11:05. Gates 15 and 16 opened.
11:07. Gates 15 and 16 closed.
11:07:30. Rising stem valve 226 closed.
11:08:30. Rising stem valve 227 closed.
11:14. Rising stem valve 210 opened half way.
11:25. Rising stem valve 210 and 211 opened all the way.
11:35. Gates 7 and 8 opened.
11:38. Gates 7 and 8 closed.
11:41. Gates 3 and 4 closed.

PERSONALS

Mr. Lawrence S. Kaiser has been appointed Superintendent of Water Works of Portland, Ore.

Mr. S. R. Fisher, Assistant City Engineer of Springfield, Ohio, has been made City Engineer, succeeding Mr. C. E. Phillips, resigned.

Mr. E. C. Bagwell, recently Assistant Engineer of the Seaboard Air Line Ry., at Norfolk, Va., has been appointed Asistant to the President, with headquarters at Norfolk.

Mr. James M. Johnson, M. Am. Soc. C. E., has resigned as Engineer of the Louisville Bridge & Iron Co., Louisville, Ky., and is now Consulting Engineer of the Illinois Central R.R.

Mr. Elijah Sniffen, a civil engineer employed by the Department of Water Supply, Gas and Electricity, Borough of Richmond, New York City, has retired after 35 years in the service of the city.

Mr. Robert C. Wood, a banker of New York City, has been appointed a member of the New York Public Service Commission, First District, succeeding Mr. John E. Eustis, whose term has expired.

Mr. Arthur A. Skeels, for the past five years Structural Engineer with Hubbell & Benes, Architects, Cleveland, Ohio, has opened offices for private practice as Structural Engineer at 4500 Euclid Ave., Cleveland.

Mr. Jerome Locke has resigned as United States Surveyor General for the District of Montana, Department of the Interior, to assume the management of the Livingston (Mont.) "Post and Enterprise."

Mr. F. C. Ewing, Jr., has been appointed Associate Professor of civil engineering at the University of the South, Sewanee, Tenn. Mr. Ewing is an engineering graduate of the University of Virginia and has been in railway work.

Mr. William D. Cantillon, General Manager of the lines east of the Missouri River, Chicago & Northwestern Ry., has resigned. Mr. Cantillon has been continuously in the service of the Chicago & Northwestern Ry. since 1878, when he began as a brakeman.

Mr. C. A. Gill, former General Master Mechanic of the Baltimore & Ohio R.R., Wheeling, W. Va., has been promoted to be Assistant Superintendent of Motive Power, with headquarters at Baltimore, Md., succeeding Mr. J. W. G. Brewer, promoted.

Prof. L. A. Waterbury, M. Am. Soc. C. E., head of the department of civil engineering, University of Arizona, has been granted a year's leave of absence, 1914-15. The department will be in charge of Assistant Professor F. C. Kelton, Assoc. M. Am. Soc. C. E.

Prof. William K. Hatt, M. Am. Soc. C. E., of the civil engineering department of Purdue University, has been appointed Engineer of the Indiana Flood Prevention Commission, which is to study and report upon flood protection works for Indiana streams.

Mr. Edward J. Govern, M. Am. Soc. C. E., formerly Division Engineer, Western division, New York State Barge Canal, is now Vice-President of the T. A. Gillespie Co., General Contractors, New York City. Previous to his Barge Canal work Mr. Govern was Superintendent of Erection for the McClintic-Marshall Construction Co.

Mr. E. H. Baldwin has been appointed Locating Engineer for the La Salle Terminal Ry., for which Isham Randolph & Co. are Consulting Engineers, Chicago, Ill. Preliminary surveys for the line from La Salle to Oglesby, Ill., have been completed. There will be three steel bridges and several reinforced-concrete structures.

Mr. Claude E. Chappell, Assoc. M. Am. Soc. C. E., recently with Dabney H. Maury, M. Am. Soc. C. E, Consulting Engineer, of Chicago, Ill, has been appointed City Manager of Big Rapids, Mich. Mr. Chappell is 35 years old and a civil engineering graduate of the University of Michigan. He was City Engineer of Benton Harbor, Mich., 1908-1910.

Mr. Clinton H. Fisk, Assoc. M. Am. Soc. C. E., Consulting Engineer, St. Louis, Mo., has been appointed Assistant Sewer Commissioner of St. Louis, at a salary of $2500 per annum. Mr. Fisk is a civil engineering graduate of Washington University, class of 1896, and for several years was Engineer of Construction of the St. Louis & San Francisco R.R.

Mr. S. G. Strickland, Assistant General Manager of the Chicago & Northwestern Ry. lines east of the Missouri River, has been promoted to be General Manager, succeeding Mr. W. D. Cantillon, resigned, as noted elsewhere. Mr. Strickland began his railway experience as a telegraph operator on the Canadian Pacific Ry. His connection with the Chicago & Northwestern Ry. began in 1908 as Asistant General Superintendent.

Mr. Frank H. Constant, M. Am. Soc. C. E., Professor of structural engineering at the University of Minnesota, has been appointed Professor of civil engineering at Princeton University, Princeton, N. J., succeeding Prof. Charles McMillan, M. Am. Soc. C. E., who has been made Professor Emeritus. Prof. Constant was born in Cincinnati, Ohio, in 1869 and graduated from the University of Cincinnati in 1891. He has been a member of the faculty of the University of Minnesota since 1895.

Col. George Washington Goethals, M. Am. Soc. C. E., Corps of Engineers, U. S. A., has been given almost unlimited authority as Governor of the Panama Canal Zone. President Wilson, by an executive order of May 16, empowers the Governor of the Panama Canal to grant pardons and reprieves for offenses against the laws and regulations, and to commute sentences and remit fines. He also may establish a parole system and prescribe regulations for the employment and welfare of the prisoners in the penal institutions in the zone.

Prof. C.-E. A. Winslow will resign as Associate Professor of biology at the College of the City of New York at the end of the present school year to become Director of the Division of Publicity and Education of the recently reorganized New York State Department of Health. Prof. Winslow's headquarters will be at the New York City offices of the State Department of Health, 25 W. 45th St. He will continue as Curator of Public Health at the American Museum of Natural History and Associate in sanitary science at Teachers College, Columbia University.

Mr. D. L. Reaburn, M. Am. Soc. C. E., Civil and Hydraulic Engineer, Los Angeles, Calif., has been appointed Locating Engineer for the Alaska Engineering Commission. Mr. Reaburn has an interesting and varied experience. It will not be his first experience in Alaska for he first went there in 1901 in the service of the U. S. Geological Survey. More recently he has been Division Engineer of the Los Angeles Aqueduct, and in 1912 and 1913 he had charge of the location surveys for a railway in Northern Patagonia for the Argentine Government (see "Engineering News," Feb. 12, 1914).

The New York, New Haven & Hartford R.R. anounces the following changes in its operating staff: Mr. W. H. Foster, former Superintendent of the Shore Line division, has been promoted to be Superintendent of the New York division, at New York City, succeeding Mr. C. H. Motsett, resigned; Mr. J. D. Gallary, former Superintendent of the Providence division, has been promoted to be Superintendent of the Shore Line division at New Haven, Conn.; Mr. R. D. Fitzmaurice, former Superintendent of the Western division, has been promoted to be Superintendent of the Providence division at Providence, R. I.; and Mr. M. D. Miller, former Trainmaster of the Providence division, has been promoted to be Superintendent of the Western division.

Lieut.-Col. William L. Sibert, M. Am. Soc. C. E., Corps of Engineers, U. S. A., former Division Engineer of the Atlantic division, Panama Canal, has been appointed to head the board of engineers to devise plans for preventing floods in Eastern China, for which $20,000,000 is to be available. The other members of the board or commission are: Messrs. Arthur P. Davis, M. Am. Soc. C. E., Chief Engineer of the United States Reclamation Service; Daniel W. Mead, M. Am. Soc. C. E., Profesor of hydraulics and sanitary engineering, University of Wisconsin, and Charles D. Jameson, M. Am. Soc. C. E., former Supervising Engineer and Architect to the Imperial Chinese Board of Foreign Affairs, who since June, 1911, has been in charge of the American Red Cross Society flood-prevention investigations in China. A preliminary report by Mr. Jameson was published in "Engineering News" of Sept. 25, 1913. When the construction work is undertaken, it is reported, Col. Sibert will be made Engineer-in-Chief.

Mr. W. J. Towne, former Engineer of Maintenance-of-way of the Chicago & Northwestern Ry., and since April, 1912, General Superintendent at Chicago, Ill., has been promoted to be Assistant General Manager of the lines east of the Missouri River, succeeding Mr. S. G. Strickland, promoted, as noted elsewhere. Mr. Towne was born in Leavenworth, Kan., Nov. 28, 1867, and he graduated from Rensselaer Polytechnic Institute in 1895. Previous to that time he had been a rodman, transitman and Assistant Engineer with the Atchison, Topeka & Santa Fé Ry. After his college course he was for four years Assistant Engineer on the New York State Canals, resigning in February, 1899, to become Assistant Engineer of Construction with the Chicago & Northwestern Ry. He was rapidly promoted to be Assistant Engineer and later Division Engineer in the maintenance-of-way department. In 1906 he was made Engineer of Permanent Improvements and from July, 1906, until April, 1912, he was Engineer of Maintenance-of-way.

Mr. James B. Wilson, Assoc. M. Am. Soc. C. E., has been appointed Chief Engineer of the Louisville (Ky.) Water Co., succeeding Mr. Theodore Leisen, M. Am. Soc. C. E., whose resignation to become Chief Engineer of the Board of Water Commissioners of Detroit, Mich., was noted in our issue of Apr. 30. Mr. Wilson is a native of Louisville, 36 years old, and a graduate of Rensselaer Polytechnic Institute, class of 1899. His first engineering work was as Assistant Engineer at a Pennsylvania cement works. Soon after he went back to Louisville, where he was employed for some time as a draftsman and Assistant Engineer of the Louisville Water Co. Later he was with the Stewart Iron Works, Cincinnati, Ohio, the Columbian Fireproofing Co., Chicago, Ill., and for three years Superintendent of the Cabrera Mines, Hostotipaquillo, Jalisco, Mexico. On his return to this country he was Resident Bridge Engineer of the new Ohio River bridge at 29th and High Sts., Louisville, and afterward Bridge Engineer of the Kentucky & Indiana Terminal R.R. Mr. Albert A. Krieger, Jun. Am. Soc. C. E., who has been Acting Chief Engineer since the resignation of Mr. Leisen, has been made Chief Assistant Engineer in charge of the filtration plant at a salary of $3000 per annum.

Dr. Henry P. Walcott has resigned as a member of the Massachusetts State Board of Health after 33 years of service. On the eve of his retirement Dr. Walcott was presented with a memorial signed by 2200 physicians of the state, in which the following record of his work in sanitation is given:

> Your connection with the board began in 1880, 33 years ago, when, after 10 years of independent existence, it had been merged with the conjoined board of Lunacy and Charity, and you were unanimously elected its health officer. At this time you served on a commission for the sanitary improvement of the Blackstone river, a precursor of your subsequent labors on similar problems.
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> It is impossible to separate your work in connection with the Board of Health from that in connection with the north and south metropolitan sewerage systems, the Charles River valley system, the Charles River estuary improvement, the metropolitan water supply and numerous other similar problems of perhaps secondary importance, such as the improvement of the Neponset River valley, of the Concord and Sudbury rivers, of the sanitary conditions as respects water-supply, sewerage and sewage disposal of many cities and towns which have been devised by the committee on water supply and sewerage of the Board of Health, of which Mr. Hiram F. Mills is chairman, and carried out in connection with its recommendations under your chairmanship of the board.
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> Since the reëstablishment of the State Board of Health in 1886, under your chairmanship, it has been the custom of the Legislature to refer all important sanitary questions to that board for investigation and advice, instead of creating special commissions, as obtains in many states. This custom, under your wise administration, has doubtless saved much money to the state, and at the same time, secured sanitary improvements recognized in all civilized countries as the best of their class.

OBITUARY

Albert Conant Buttrick, a pioneer railway civil engineer of Massachusetts, died May 20, at Worcester, Mass., aged 84 years.

A. A. Sinclair, Superintendent of Bridges and Buildings, Duluth, Missabe & Northern R.R., was struck by a locomotive and killed, May 14, at Duluth, Minn.

Louis Herry, of New Brounfels, Tex., Superintendent of the Midland Engineering & Construction Co., died May 9. He built several bridges and viaducts in Texas.

George Millard, Secretary of the Citizens Water Co., Burlington, Iowa, and for many years connected with the Murray Iron Works Co., died May 4, aged 45 years. He is survived by a widow.

Martin L. Gardner, M. Am. Soc. C. E., Assistant Engineer, Pennsylvania R.R., died at his home in Newark, N. J., May 14. He was born in Newark in 1861 and studied at Columbia University. He had been employed in the engineering department of the Pennsylvania R.R. for many years, specializing in pier and harbor work.

Charles Barnes Hoadley, M. Am. Inst. M. E., a graduate of Sheffield Scientific School, Yale University, class of 1903, was one of several Americans who have recently met violent deaths in Mexico. Mr. Hoadley was Superintendent of the El Favor mines at Hostotipaquillo, Jalisco, Mexico. The mine was recently set upon by bandits, who murdered Mr. Hoadley and another American, G. E. Williams, of Philadelphia, Penn.

J. Roy Dillon, a former railway civil engineer, died May 18, at Mineral Wells, Tex. He was 44 years old. He was born in Lexington, Va. About 18 years ago he went to Galveston, Tex., as a civil engineer and for several years was employed by the Gulf, Colorado & Santa Fé Ry. Later he was Vice-President and General Manager of the Houston Belt & Terminal Ry., and was subsequently connected with other railway and terminal companies.

Horace A. Cook, Assoc. M. Am. Soc. C. E., Structural Engineer, of the firm of Streiz, Cook & Smart, Phœnix, Ariz., died May 16, at his home in Phœnix. Since settling in Phœnix in 1909 Mr. Cook had charge of much building work and also of street pavement work. Before going to Phœnix he had been in engineering work in Seattle, Wash. He was a graduate of Valparaiso University and Purdue University, in Indiana. He is survived by a widow and a daughter.

Robert Crichlow Dewar, Engineer for the McHarg-Barton Co., on the construction of a reinforced-concrete arch bridge over the Norwalk River at South Norwalk, Conn., was drowned May 21, by the collapse of a coffer-dam, the pumping out of which he was superintending. The details of the accident are given on another page of this issue. Mr. Dewar was a native of East Orange, N. J., and graduated in civil engineering at Cornell University in 1908. He was 26 years old.

Leon L. Gay, Assoc. M. Am. Soc. C. E., died May 6, in Montreal, Que., at the age of 35 years. Mr. Gay was born in Barton Landing, Vt., graduated from the Sheffield Scientific School, Yale University, in 1901, and immediately went to Idaho to become Assistant Engineer on the construction of a power plant at Horseshoe Bend on the Payette River. After the completion of this plant he went into irrigation engineering work. He was in the U. S. Reclamation Service from 1903 to 1908, acting as Assistant Engineer on the construction of the Minidoka dam on Snake River, the original Jackson Lake Dam in Wyoming, and the Boise River Dam, near Boise, Idaho. In 1908 he resigned from the Reclamation Service and became Engineer for a large irrigation enterprise in the Sacramento Valley in California. In 1909 he contracted tuberculosis and has been fighting this disease and its after-effects for the past five years. At the time of his attack he was considered as one of the most promising young engineers in the country in the line of irrigation engineering and had already done some very important and successful work. He was well known and universally respected by irrigation engineers of the West. For these reasons his misfortune and death are peculiarly sad. [Contributed by F. C. Horn, M. Am. Soc. C. E., Consulting Engineer, Boise, Idaho.]

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 2-5. Annual convention at Baltimore, Md. Secy., Chas. W. Hunt, 220 W. 57th St., New York City.

MASTER CAR BUILDERS' ASSOCIATION.
June 10-12. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 1112 Karpen Building, Chicago, Ill.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
June 11-13. Annual meeting at Toledo, Ohio. Secy., Frank E. Sanborn, Columbus, Ohio.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 15-17. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 1112 Karpen Building, Chicago, Ill.

AMERICAN SUPPLY AND MACHINERY MANUFACTURERS' ASSOCIATION.
June 15-17. Annual convention at White Sulphur Springs, W. Va. Secy., F. D. Mitchell, Woolworth Building, New York City.

SOUTHWESTERN WATER-WORKS ASSOCIATION.
June 15-17. Annual meeting at Tulsa, Okla. Chairman of Arrangements, L. L. Ballard, Tulsa, Okla.

TRAIN DISPATCHERS' ASSOCIATION OF AMERICA.
June 16-19. Annual convention at Jacksonville, Fla. Secy., John F. Mackie, 7122 Stewart Ave., Chicago, Ill.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 16-19. Spring meeting at St. Paul and Minneapolis, Minn. Secy., Calvin W. Rice, 29 W. 39th St., New York City.

AMERICAN INSTITUTE OF CHEMICAL ENGINEERS.
June 17-20. Sixth semi-annual meeting at Troy, N. Y. Secy., J. C. Olsen, Polytechnic Institute, Brooklyn, N. Y.

SOCIETY OF AUTOMOBILE ENGINEERS.
June 23-26. Annual convention at Cape May, N. J. Chairman of Arrangement Committee, Arthur B. Cummer, 1790 Broadway, New York City.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 23-26. Thirty-first annual convention at Detroit, Mich. Secy., F. L. Hutchinson, 29 W. 39th St., New York City.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 23-26. Annual meeting at Princeton, N. J. Secy., Henry H. Norris, 29 W. 39th St., New York City.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 30-July 4. Seventeenth annual meeting at Atlantic City, N. J. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

AMERICAN SOCIETY OF ENGINEERS, ARCHITECTS AND CONSTRUCTORS.
July 3-4. Convention at Brighton Beach, L. I. Secretary, T. Hugh Boorman, 520 E. 20th St., New York City.

AMERICAN SOCIETY OF ENGINEERING CONTRACTORS.
July 4. Convention at Brighton Beach. Secretary, Julius Wemlinger, 11 Broadway, New York City.

Water-Works Manufacturers' Association—During the recent convention in Philadelphia, of the American Water-Works Association, the water-works manufacturers elected the following officers: President, Oscar B. Mueller, of Sarnia, Ont.; Vice-President, Fred S. Bates, Troy, N. Y.; Treasurer, Robert E. Milligan, New York City. The secretaryship will be filled by appointment.

Texas Association of Members of the American Society of Civil Engineers—Among the speakers at the summer convention recently held at Austin, Tex., by the newly organized Texas Association of Members of the American Society of Civil Engineers, were the following: James C. Wonders, Washington, D. C.; R. D. Parker, Dallas; C. M. Chamberlain, Dallas. It was decided to hold the fall convention at Dallas, at which time officers for the ensuing year will be elected.

Baltimore Section of American Society of Civil Engineers—A Baltimore section of the American Society of Civil Engineers was recently formed. On May 6, at Arundel Hall, Baltimore, the following officers were elected: President, J. E. Greiner; First and Second Vice-Presidents, Francis Lee Stuart, Chief Engineer, Baltimore & Ohio R.R., and L. H. Beach, Colonel, Corps of Engineers, U. S. A.; Secretary-Treasurer, Harry D. Williar, Jr., Assistant Engineer, Paving Commission, Baltimore, Md. The directors are H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, A. W. Thompson. The first regular meeting was held May 20.

Engineers' and Architects' Club of Louisville, Ky.—At a meeting, on May 19, Fred S. Shafer, Drainage Engineer, U. S. Department of Agriculture, read a paper before the club on the subject of "Scientific Principles of Drainage." Mr. Shafer's paper contained the following information:

Drainage investigation is a branch of the Bureau of Experiment Stations, United States Department of Agriculture, which has been in existence about ten years, and during which time has had appropriations totaling about $700,000. Plans have been prepared during this time for the reclamation of about 10,000,000 or 12,000,000 acres of swamp, wet or overflowed lands. These lands are at present worthless, from an agricultural standpoint. The total estimated area of swamp, wet and overflowed lands in the United States, from statistics prepared in 1908, by drainage investigations, was approximately 79,000,000 acres.